CHAPTER 5

PROJECT DESIGN

Section I. Spillway Design

5-1. General. Navigation dams can be relatively high structures, such as those on the Columbia and Snake Rivers, in which cases the spillway should be designed in accordance with procedures described in EM 1110-2-1603. However, most navigation dams are low-head structures. Their basic purpose is to provide adequate depths for navigation during low-flow periods and to offer minimum resistance to high flows. This chapter concentrates on the design of spillways for low-head dams. The following guidance is mainly a result of analysis of specific low-head navigation projects. A definition sketch is given in Figure 5-1 and symbols are defined in Appendix B. An example design is provided at the end of this chapter.

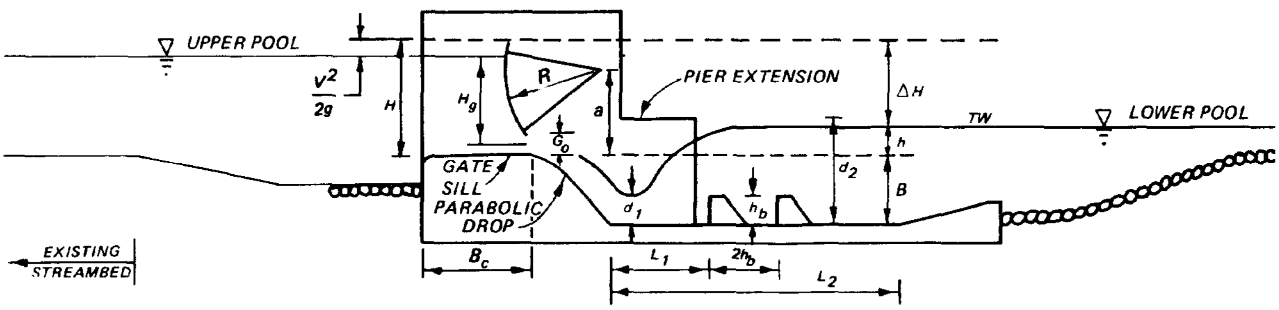


Figure 5-1. Definition sketch of typical navigation dam

5-2. Crest Design.

a. General. Since the project is planned to offer minimum resistance to flood flows, the fixed portion of the spillway must occupy only a small part of the cross section of the river channel. Thus a gate sill with its elevation at or near the elevation of the streambed is required and damming during low flows must be accomplished by movable gates. The lower the head on the crest, the lower the unit discharge. This results in a longer crest but lesser requirements for the stilling basin and downstream channel protection. Conversely, the higher the head on the crest, the higher the unit discharge. This results in a shorter crest length but greater requirements for the stilling basin and downstream channel protection. Many low-head navigation dams operate under highly submerged flow conditions. The discharge coefficients for a low, submerged, broad-crested weir are close to those for a similar low, submerged ogee crest. With a low gate sill an ogee crest may not provide sufficient space for operating gates and bulkheads. Thus, for these reasons, a broad-crested weir is often indicated and structural requirements usually dictate the width of the crest to be approximately the same as the damming height of the gates. For structures that do not operate under submerged flow conditions, an ogee crest is often used to improve efficiency of the spillway. EM-1110-2-1603 provides guidance for design of ogee crests. The remainder of paragraph 5-2 addresses crest design for broad-crested weirs.

b. Upstream Face. Although a vertical upstream face slope has been used on most low-head navigation dams having a broad-crested weir, other slopes can be used. Based on an analysis of the data presented in item 3 of Appendix A, the minimum radius connecting the upstream face with the horizontal portion of the broad-crested weir should be as follows:

Head, feet	Radius, feet
<20	3
20-30	4
30-40	5
40-50	6

c. Downstream Face for Nonsubmersible Gate Spillway. The downstream face of the weir can be shaped so that flow under partially opened gates will adhere to this face of the weir and thus move to the floor of the stilling basin where it can be dispersed by baffles and/or the end sill. If the downstream face breaks away from the weir crest too sharply, the nappe will separate from the weir, and an eddy in a vertical plane will form under the nappe in the upstream portion of the stilling basin. Under certain tailwater conditions, this eddy will force the nappe upward and then it will dive through the tailwater and attack the exit channel downstream of the stilling basin. This undesirable type of action, known as an undulating jet with a free nappe, generates severe surface waves. Of course, economics dictates that the horizontal extent of the downstream face of the weir be minimum. In item 6 of Appendix A, tests are described wherein it was established that the downstream face of the weir should be parabolic based on the trajectory of a free jet, A free jet leaving the horizontal weir crest will follow the path:

$$X^2 = \frac{2V_o^2 Y}{g} \tag{5-1}$$

where

X,Y = horizontal and vertical coordinates

V_o = initial free jet in feet per second (ft/sec) = $\sqrt{2gH}$

g = acceleration due to gravity in ft/sec^2

H = upper pool elevation, crest elevation

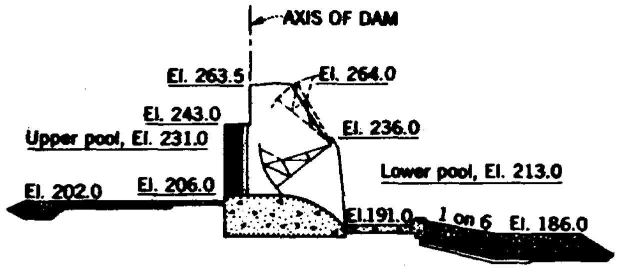
However, based on item 6 of Appendix A, the nappe will adhere to the downstream face if V is the theoretical velocity resulting from only one-third of the actual head. Thus, if the upper pool is 36 feet above the weir crest (H = 36 feet), V_o for determination of the shape of the downstream face of the weir should be based on a head of only 36/3 or 12 feet. That is, $V_o = \sqrt{2g(12)}$ = 27.8 ft/sec; and the equation for the downstream face should be about $X^2 = 48Y$. Since the range of data used to develop this relation is limited, the steepest trajectory that should be used is $X^2 = 40Y$. For heads greater than 40 feet, model testing is required. Using one-third of the head on the crest in Equation 5-1 results in a downstream face shape which is close to that resulting from the procedure used for high spillways (presented in EM 1110-2-1603). The techniques presented in EM 1110-2-1603 can be used for heads greater than 40 feet. The trajectory resulting from using one-third of the head on the crest is the steepest that can be used without severe negative pressures occurring on the downstream face; flatter trajectories can be used. The parabolic trajectory continues to the stilling basin floor unless terminated by a constant slope which may be desired for ease of construction. A slope of 1V:1H was used below the parabolic trajectory in the investigation of pressures on the downstream face of the crest (Item 6). Examples of different crests are shown in Figure 5-2. Downstream faces having “steps” have been used on Mississippi River Locks and Dams Nos. 5A, 6, 7, 8, and 9. These structures have relatively small differentials (5.5 to 11.0 feet) between upper and lower pool elevation.

d. Downstream Face, Submergible. Submergible tainter gates are used to pass ice over the top of the gates. As shown in Figure 5-3, submersible tainter gates can be either the “piggyback” type or-those in which the crest shape allows the bottom of the tainter gate to drop below the flat portion of the crest. The piggyback type uses the parabolic trajectory given in (c) above. Two examples of the downstream crest shape for the 2nd type of submergible tainter gate are shown in Figure 5-3. Gate bays for submergible gates should not be so wide that undesirable gate vibrations develop.

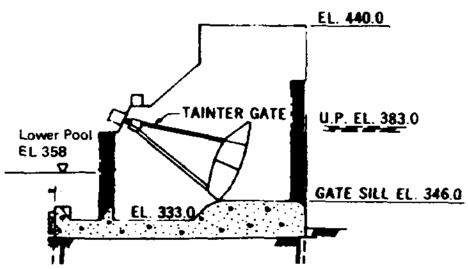
e. Intersection of Downstream Spillway Face and Stilling Basin Floor. Toe curves at the intersection of the downstream spillway face and the stilling basin floor are not widely used in low-head navigation dams. Guidance for toe curve pressures below ogee crests is given in HDC 122-5.

f. Crest Pressures, Velocities, and Water-Surface Profiles. For most low-head navigation dams, spillway velocities are relatively moderate because of tailwater submergence effects. Under normal spillway operations, all the gate openings would be balanced and maximum velocities would occur at small gate openings when the effective head is high and tailwater level is low. The latest design policies require that under emergency conditions, any one gate can be fully opened without causing severe erosion damage to the downstream scour protection measures. Flow velocities and pressures should be determined for both of these operational conditions. The velocities are needed to assign appropriate tolerances for construction of the spillway surfaces. Pressures resulting from these velocities are needed to ensure against cavitation conditions and also to determine the uplift forces needed by structural designers to check the spillway stability. Crest pressures and water-surface profiles have not been measured for a wide range of heads, gate openings, approach elevation, apron elevations, etc. Available information is given in item 6 of Appendix A and shown in Figures 5-4 and 5-5 for water-surface profiles and pressures, respectively.

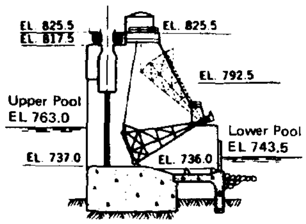
5-3. Spillway Capacity for High-Head Dams. Spillways for high-head navigation dams are generally designed with adequate capacity to pass the PMF flows. At this condition, all-flows-would still be limited to the spillway section; adjacent concrete or embankment structures would have adequate freeboard to



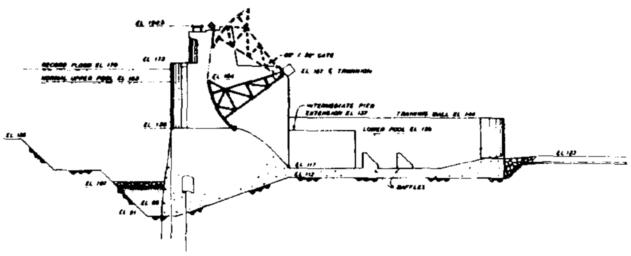
DAVID D. TERRY LOCK & DAM (NO.. 6)
(ARKANSAS RIVER)



CANNELTON LOCKS & DAM
(OHIO RIVER)

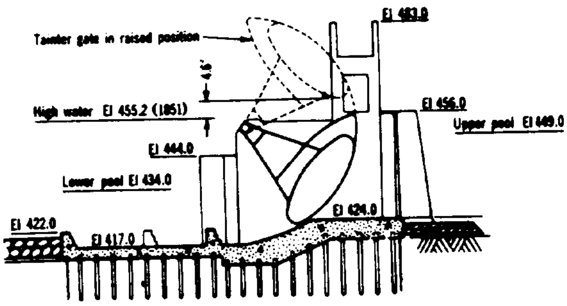


MAXWELL LOCK & DAM
(MONONGAHELA RIVER)

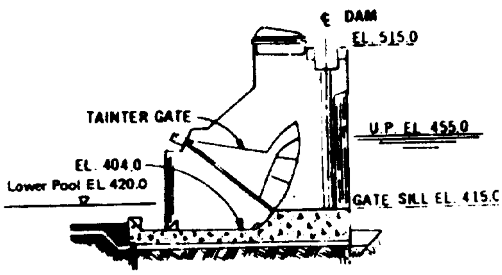


COLUMBUS LOCK & DAM
(TOMBIGBEE RIVER)

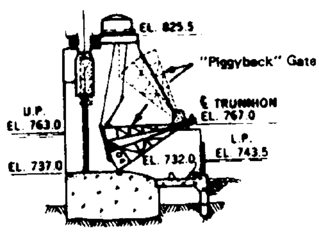
Figure 5-2. Examples of crests, nonsubmergible gates



LOCK & DAM 24
(MISSISSIPPI RIVER)



MARKLAND LOCKS & DAM
(OHIO RIVER)



MAXWELL LOCK & DAM
(MONONGAHELA RIVER)

Figure 5-3. Examples of crests, submergible gates

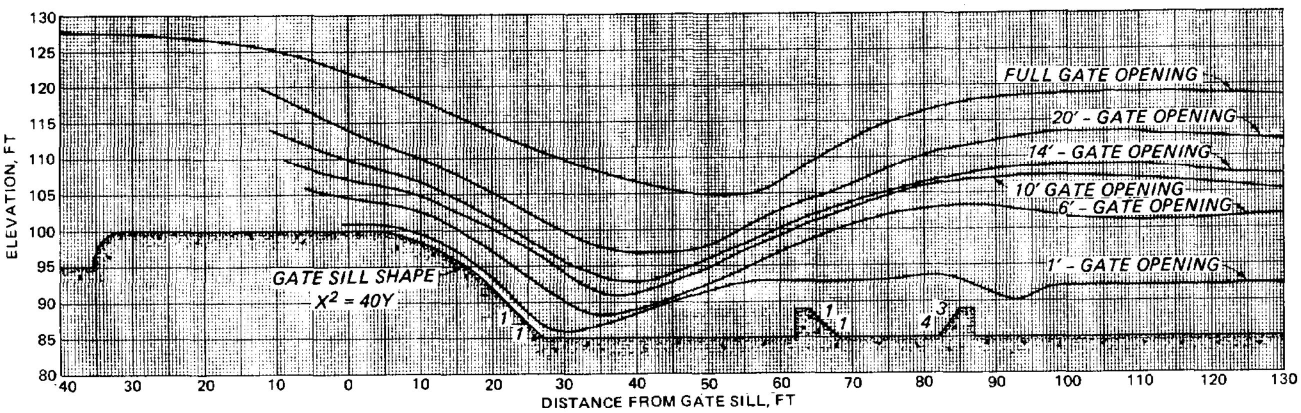


Figure 5-4. Water-surface profiles (from item 6, Appendix A)

prevent overtopping of these structures. In some cases, stilling basin designs would be based on the PMF condition, but in other cases tailwater buildup for this discharge would drown out the hydraulic jump and the design would be based on some lesser discharge condition. Reference EM 1110-2-1603 for determining spillway capacity for high-head dams.

5-4. Spillway Capacity for Low-Head Dams. Typically, low-head navigation dams are designed to pass flood flows utilizing not only the main spillway section normally located within the river channel but also supplemental spillways located across the overbanks and even the lock access road and esplanade areas. The width and potential carrying capacity of the overbanks will affect the main spillway capacity. However, the objective in sizing the main spillway is to minimize the headwater-tailwater differential at the time flood stages exceed the riverbanks, extend out into the overbank areas, and begin overtopping the uncontrolled spillways. The smaller this head differential, the less will be flood stage increases over preproject conditions, and the simpler will be the scour protection measures required for the overbank uncontrolled spillway sections. These head differentials can be kept low by providing a main spillway capacity roughly equivalent to the natural river capacity at the project design flood. Providing this much capacity can be difficult on smaller rivers because the navigation lock must be prominently located within the main river channel to provide safe lock approach conditions. Consequently, low-flow dam spillways frequently extend well into the bank line opposite the lock, unless the lock is located within a navigation canal separated from the natural river. Locating the spillway in a bypass canal is another means of reducing the head differential.

a. Spillway Crest Elevation. Low-head, gated spillways typically have crest elevations set near the riverbed elevation to maximize capacity. Of course, riverbed elevations generally vary across the proposed spillway section. Furthermore, bed elevations in alluvial rivers vary with discharges. An understanding of these alluvial characteristics during flood conditions is required to select the optimum crest elevation. If selected too high, the spillway will be wider than necessary. If selected too low, the discharge control will shift from the spillway crest to an approach channel section when

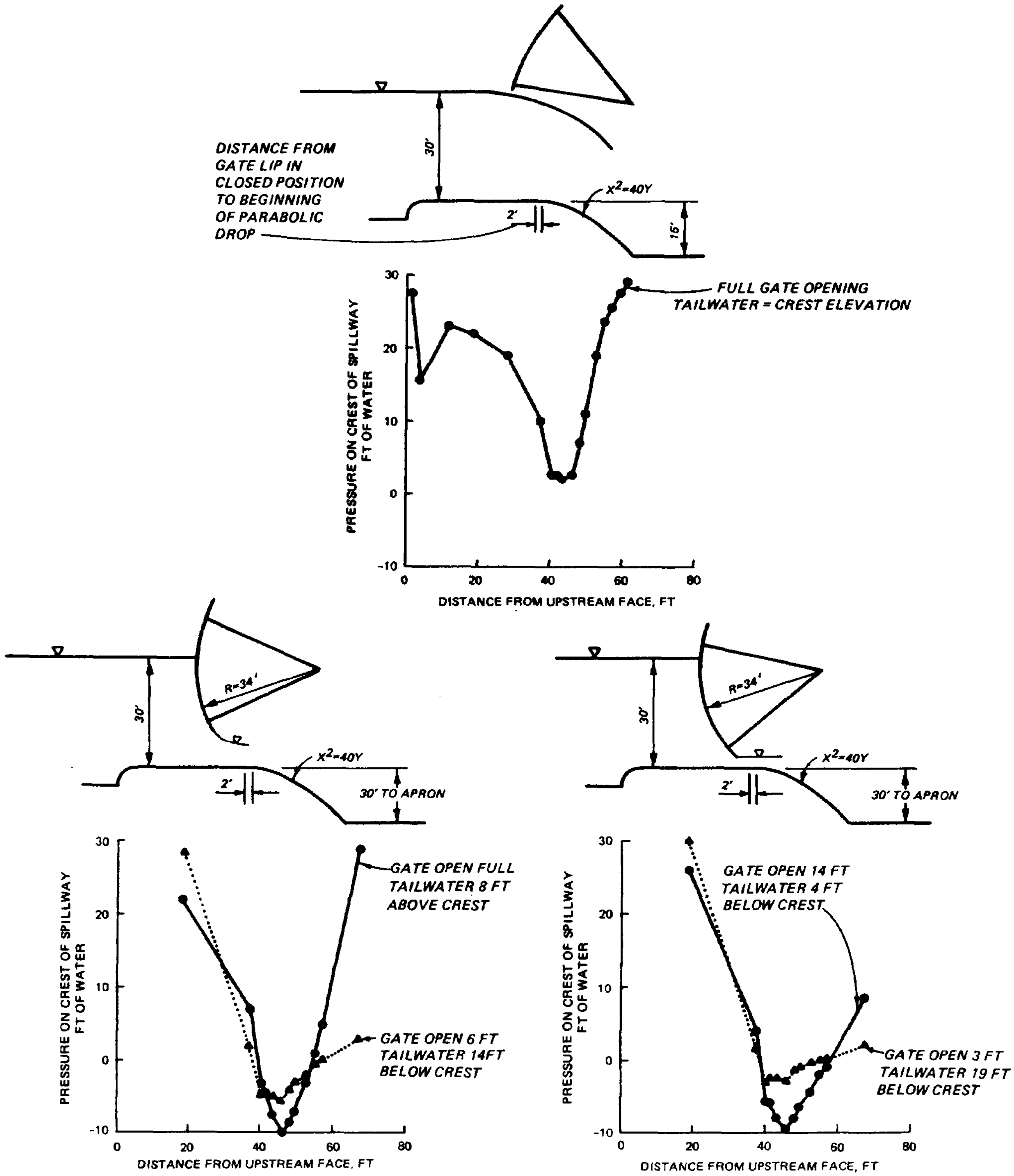


Figure 5-5. Pressures on crest for various gate openings (from item 6, Appendix A)

the gates are fully opened; the spillway gates will be higher than necessary; the spillway structural stability will be more difficult to attain; and during low-flow periods sediment will deposit on the spillway thereby hampering gate operations and increasing wear and tear of the gates. At Lock and Dam 4 on the Arkansas River, the spillway crest was set at two elevations with the deeper section next to the lock and the higher section at the opposite bank line where under preproject conditions sediments normally deposited, After over 15 years of operation, the benefits of the stepped crest are considered negligible, and a constant crest elevation would be recommended. The stilling basin design for multilevel crest elevations is complex.

b. Overbank Crest Elevation. The spillway crest elevations of uncontrolled overbank sections are generally set as close to the natural groundline as possible to best utilize the natural flow capacity of the overbank areas. However, the overbank spillway should normally be at least three feet above the navigation pool elevation to allow for pool regulation variations, wind setup, and wave runup heights. One exception would be the crest height at a navigation bypass section that is normally just one foot above the navigation pool level.

5-5. Pool-Tailwater Relationships. The size of the spillway (both horizontal and vertical) affects pool and tailwater elevations. Three general cases can be identified.

a. Case 1. The dam is of sufficient height that the spillway is not submerged by tailwater for any discharge.

b. Case 2. The height of the dam is such that the spillway operates continuously or intermittently submerged, but open-river conditions will not obtain at any time.

c. Case 3. The height of the dam is such that the spillway operates continuously or intermittently submerged with open-river conditions sometimes.

The pool and tailwater elevation regimes (see Figure 5-6) resulting from a particular project (particularly pool elevations) can affect numerous related factors such as the extent of real estate flooded, groundwater table, levee heights, dam and lock wall heights, number and extent of relocations, navigation pass velocities, etc. Determination of spillway design in relation to these factors is complex, but in general high, narrow spillways are spillway cost-effective, while low, wide spillways reduce the costs associated with high pool elevations. Sufficient spillway sizes should be studied to optimize overall project costs. Cases 2 and 3 are the most complex due to spillway submergence.

5-6. Pool Elevations. The complexity of approach flow and interaction with locks, dams, overflow sections, nonoverflow embankments, and spillway submergence make accurate pool elevation determination difficult. This is particularly true when flow approaches spillways at an angle. The d’Aubuisson (see paragraph 5-7) or Kindsvater and Carter formulas can be used for an approximate pool elevation estimate during preliminary submerged spillway design studies (see item 32). However, hydraulic models will usually be needed to obtain an estimate of pool and tailwater elevations suitable for detailed design. Computations should be made for the design flood with all gates fully opened and for all operating conditions to establish the maximum upstream pool and backwater profile. Pool elevations and backwater profiles associated with recurrence interval should also be computed to evaluate real estate, relocations, and other pertinent factors. Some Corps Districts have successfully used the special bridge routine in the HEC-2 backwater program to make these computations.

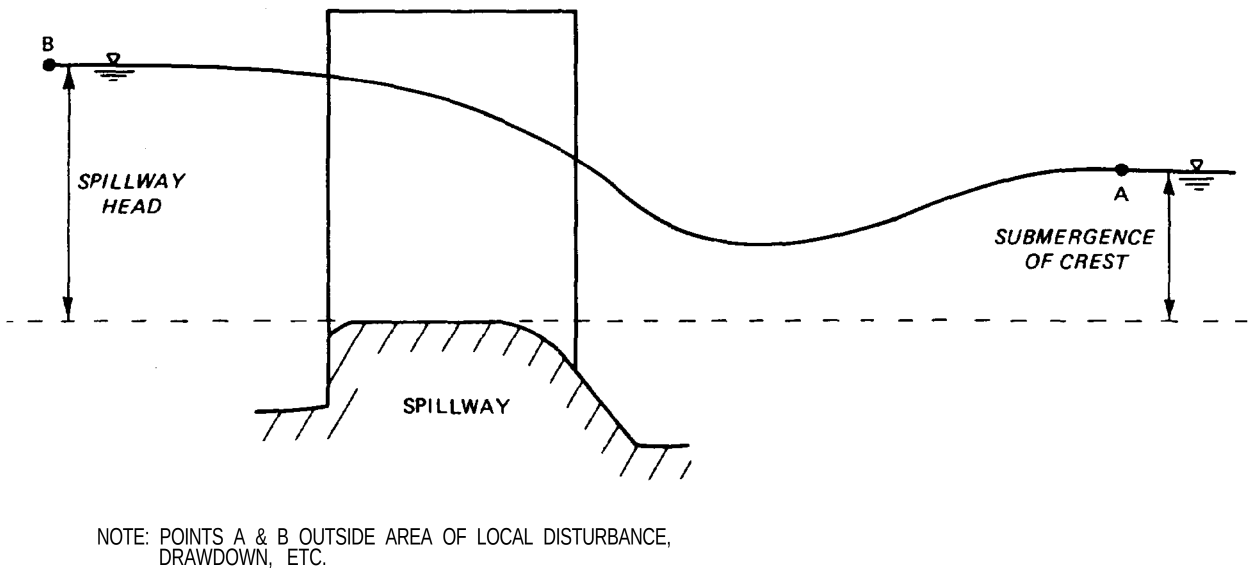


Figure 5-6. Spillway head/submergence

5-7. Discharge Rating Curves for Gated, Broad-Crested Weirs.

a. General. Discharge rating curves are needed for project design and operation. Low-head navigation structures have four possible regimes of flow that result from the effects of the gates and the effects of tailwater on the amount of discharge through the structure. The four regimes are discussed in the following paragraphs and shown in Figure 5-7. Discharge coefficients for low-head navigation dams have been developed mainly for tainter gates. Reference EM 1110-2-1603 for discharge rating of unsubmerged vertical gates or discharge rating of ogee crests. Sufficient data are not available to define the effects of different pier lengths and nose shapes. Results from item 6 of Appendix A comparing the ogival and semicircular shapes showed no significant difference for the highly submerged broadcrested weir. Preliminary curves are usually computed from established analytical equations. Physical and mathematical model studies of project facilities frequently include tests to verify both spillway rating curves and flood flow distributions between river channel and overbanks. Model and prototype data from other projects with similar spillway designs are often valuable in refining rating curves. Commonly used equations for preliminary rating curve computations under various spillway

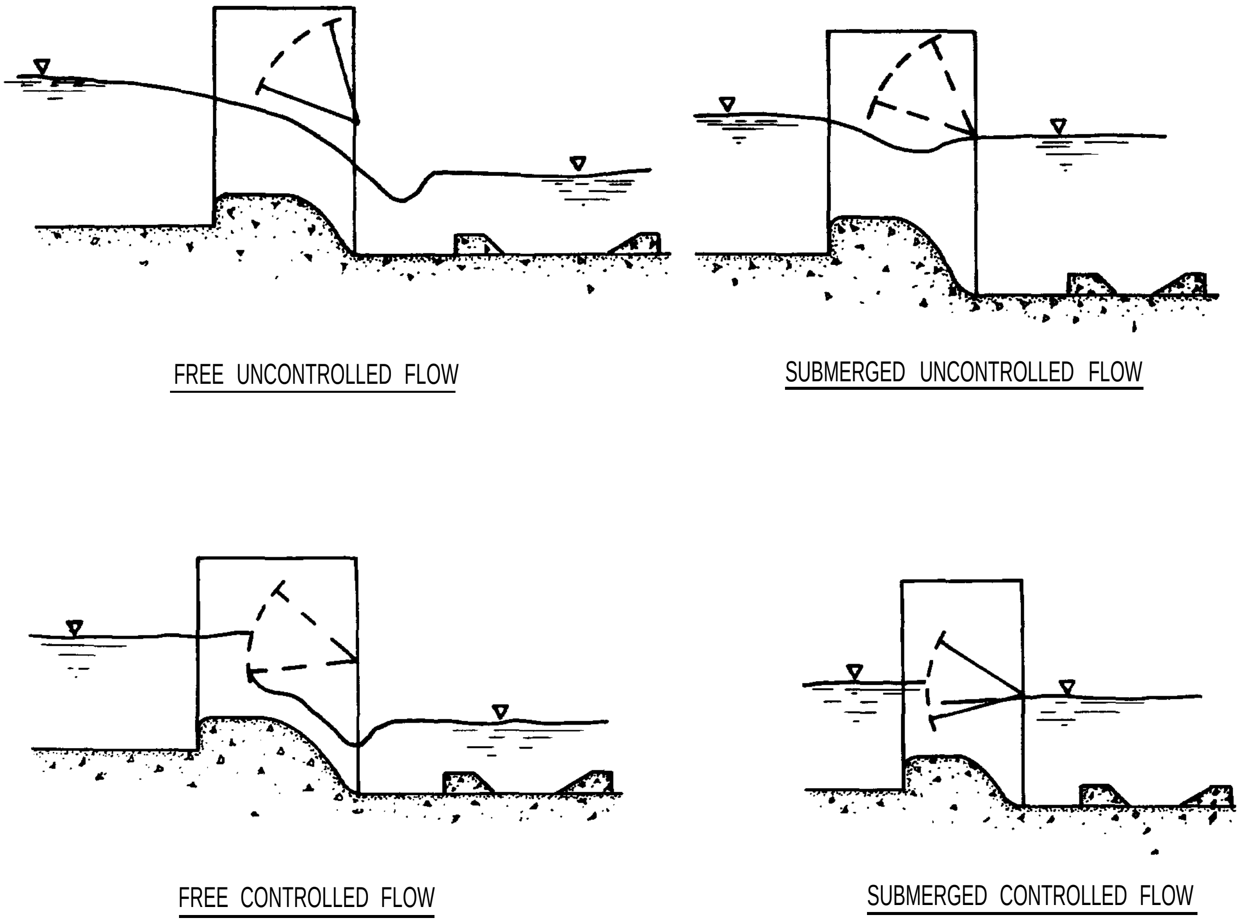


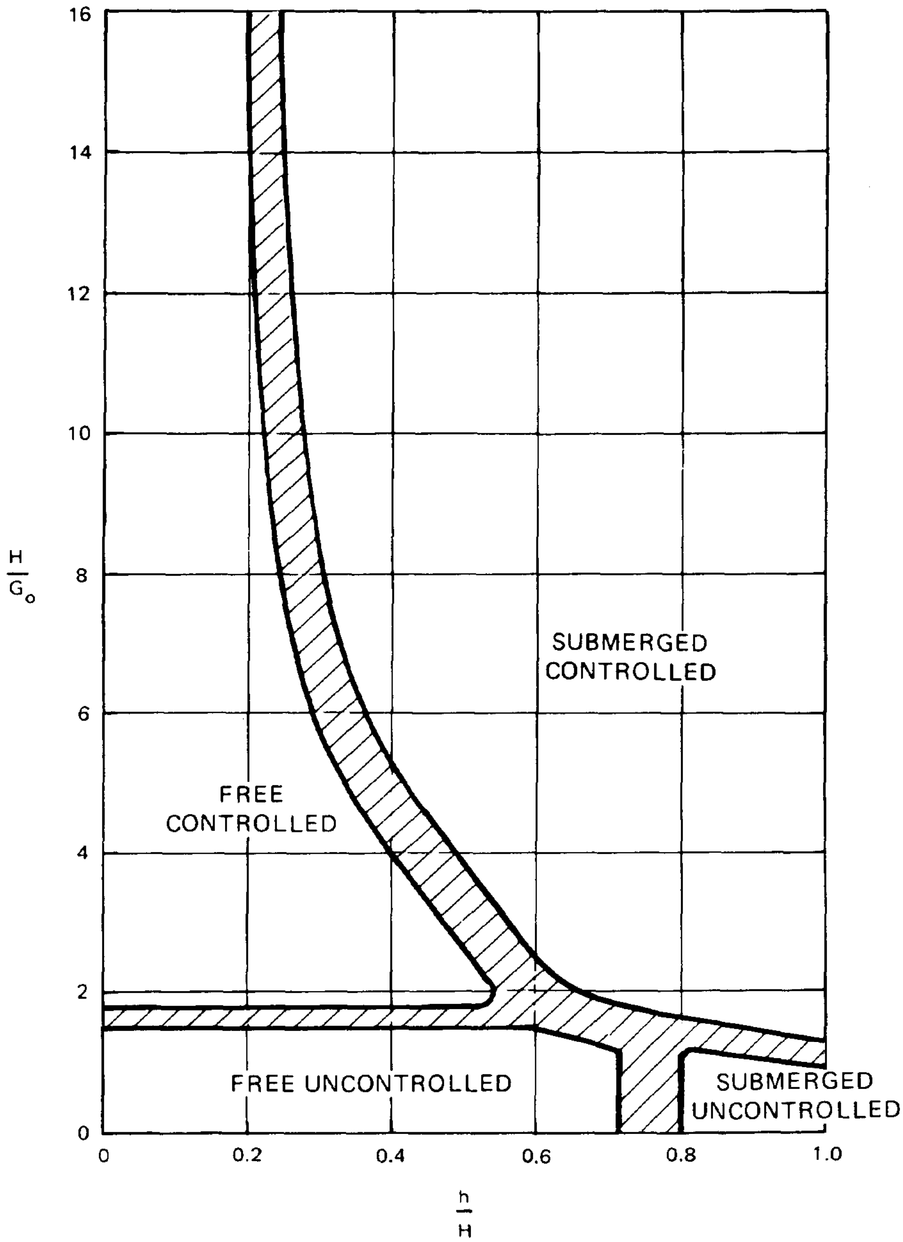
Figure 5-7. Four flow regimes

conditions are presented. A computer program was developed in the Pittsburgh District for discharge rating of navigation dams and is presented in item 22 of Appendix A.

b. Determining Flow Regime. Figure 5-8 gives guidance to determine the flow regime given headwater H, tailwater h, and gate opening G_o (definition sketch in Figure 5-1).

c. Free Uncontrolled Flow. For this flow regime the gates are fully opened and the upper pool is unaffected by the tailwater. The standard weir equation

$$Q = C_F L H^{3/2} \tag{5-2}$$



NOTE: CROSS-HATCHED AREAS REPRESENT TRANSITION ZONES
FULLY OPENED GATE EQUIVALENT TO $H/G_0 = 0$

Figure 5-8. Flow regime based on headwater, tailwater, and gate opening is applicable and free uncontrolled flow discharge coefficients versus (Head/Breadth of Crest) from item 22 of Appendix A are shown in Figure 5-9. This curve should be used with caution above $H/B_c = 1.5$. No correction for pier effects is recommended with these coefficients. Crest length should be reduced for abutment effects by the equation

$$L_{effective} = L_{actual} - 2KH \tag{5-3}$$

Since the discharge coefficients presented in Figure 5-9 already account for pier effects, the abutment contraction coefficient K should be about one-half of the value selected from HDC Chart 111.

d. Submerged Uncontrolled Flow. For this flow regime, the gates are fully opened and the discharge is reduced by tailwater conditions. Two procedures are available for determining discharges for uncontrolled spillways under submerged conditions.

(1) Discharge over a submerged weir can be expressed by the equation :

$$Q = C_S LH^{3/2} \tag{5-4}$$

C_s from model data is shown to vary with h/H. Results from item 22 of Appendix A show that discharge coefficients for this flow regime are not significantly affected by stilling basin apron elevation. Figure 5-10 presents recommended submerged uncontrolled flow discharge coefficients as a function of h/H. These coefficients were developed from a large number of model investigations.

(2) Preliminary rating curves for low-head dams under submerged uncontrolled flow conditions can be computed by the d' Aubuisson equation

$$Q = KLh \sqrt{[2g (H - h) + V^2]} \tag{5-5}$$

where

K = spillway coefficient of contraction

L = crest length = number of bays times the bay width, ft

V = spillway approach velocity, ft/sec

H, h = see Figure 5-1

Suggested K values vary with spillway bay width as follows:

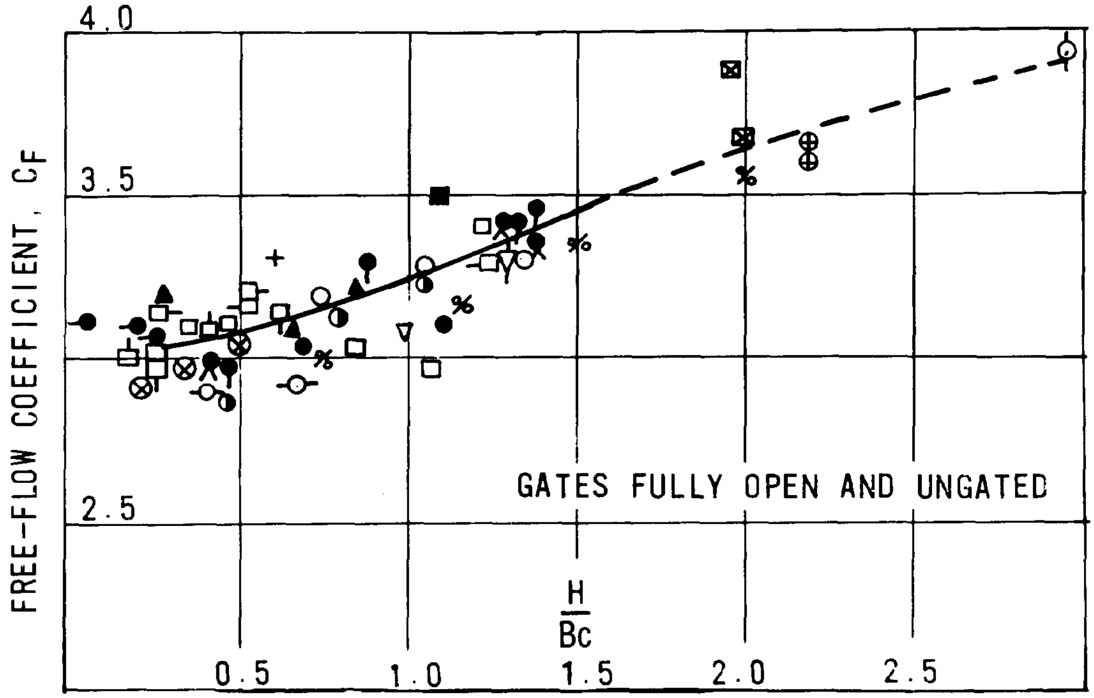


Figure 5-9. Free flow discharge coefficient for uncontrolled flow over a broad-crested weir

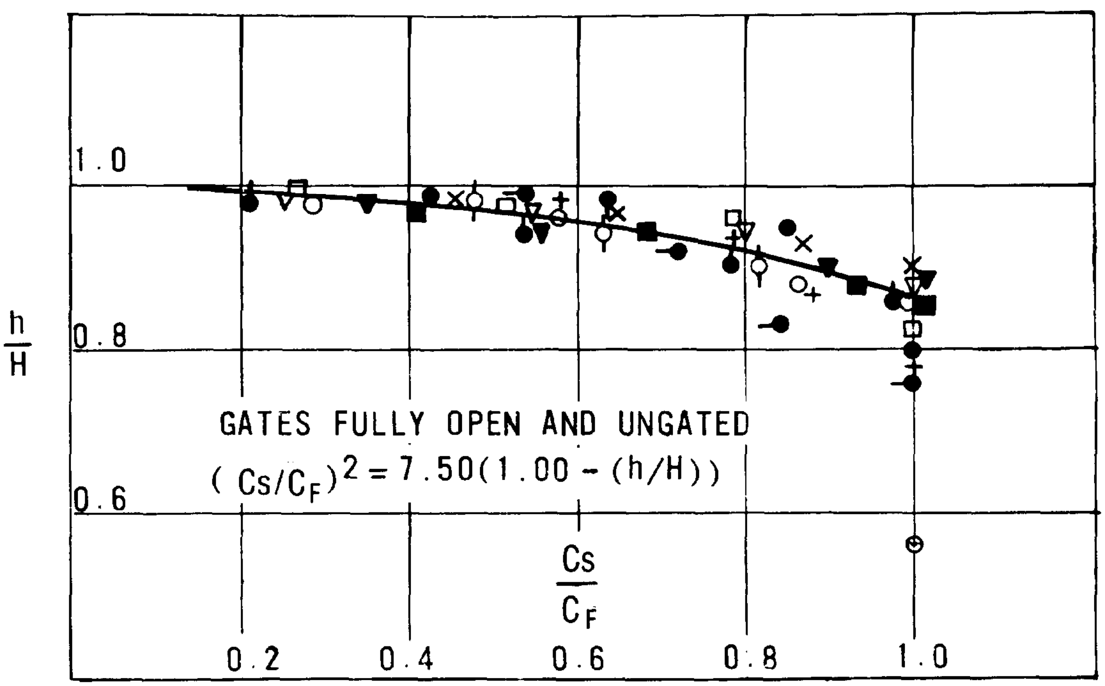


Figure 5-10. Submerged uncontrolled discharge coefficient for broad-crested weir

Bay Width, feet	K
40	0.80
50	0.85
60	0.90
110	0.95

These coefficients were developed from experience with prototype structures. Several different methods exist for predicting discharge for submerged uncontrolled flow. These include the methods presented above and HDC 111-4, items 6 and 32 in Appendix A. These methods do not give similar results.

e. Free Controlled Flow. For this flow regime, the gates are partially open and the upper pool is unaffected by the tailwater. Discharge is controlled by the gate opening and two approaches are available for determining discharge.

(1) Results from item 22 of Appendix A shown in Figure 5-11 present the free controlled flow discharge coefficient as a function of gate opening, gate radius (R), trunnion height above crest (a), and gross head on the gate. Figure 5-11 is applicable to heads and gate openings less than 30 and 14 ft, respectively. The applicable equation is

$$Q = C_g L G_O \sqrt{2gH} \tag{5-6}$$

(2) For conditions outside the range covered in (1), a comprehensive treatment of the effects of gate location and geometry on discharge for free controlled flow is presented in HDC 320-4 to 320-7. Caution should be used because the equations and symbols are not the same in the two methods.

f. Submerged Controlled Flow. For this flow regime, the gates are partially open and the upper pool is controlled by both the submergence effect of the tailwater and the gate opening. The applicable equation is

$$Q = C_{gs} L h \sqrt{2g\Delta H} \tag{5-7}$$

The submerged controlled discharge coefficient C_{gs} as a function of h/G for various apron elevations is given in Figure 5-12. See item 22 in Appendix A for a similar method for submerged controlled flow that has been used in the computer program referred in paragraph 5-7 (a).

g. Rating Curve Accuracy.

(1) Discharge Coefficients. Spillway rating curves as computed by the above equations require verification for final designs. Significant errors are possible because of the unique approach conditions at proposed projects. Although data comparing model-prototype rating curves are rare, such information derived from similar existing projects would be valuable for

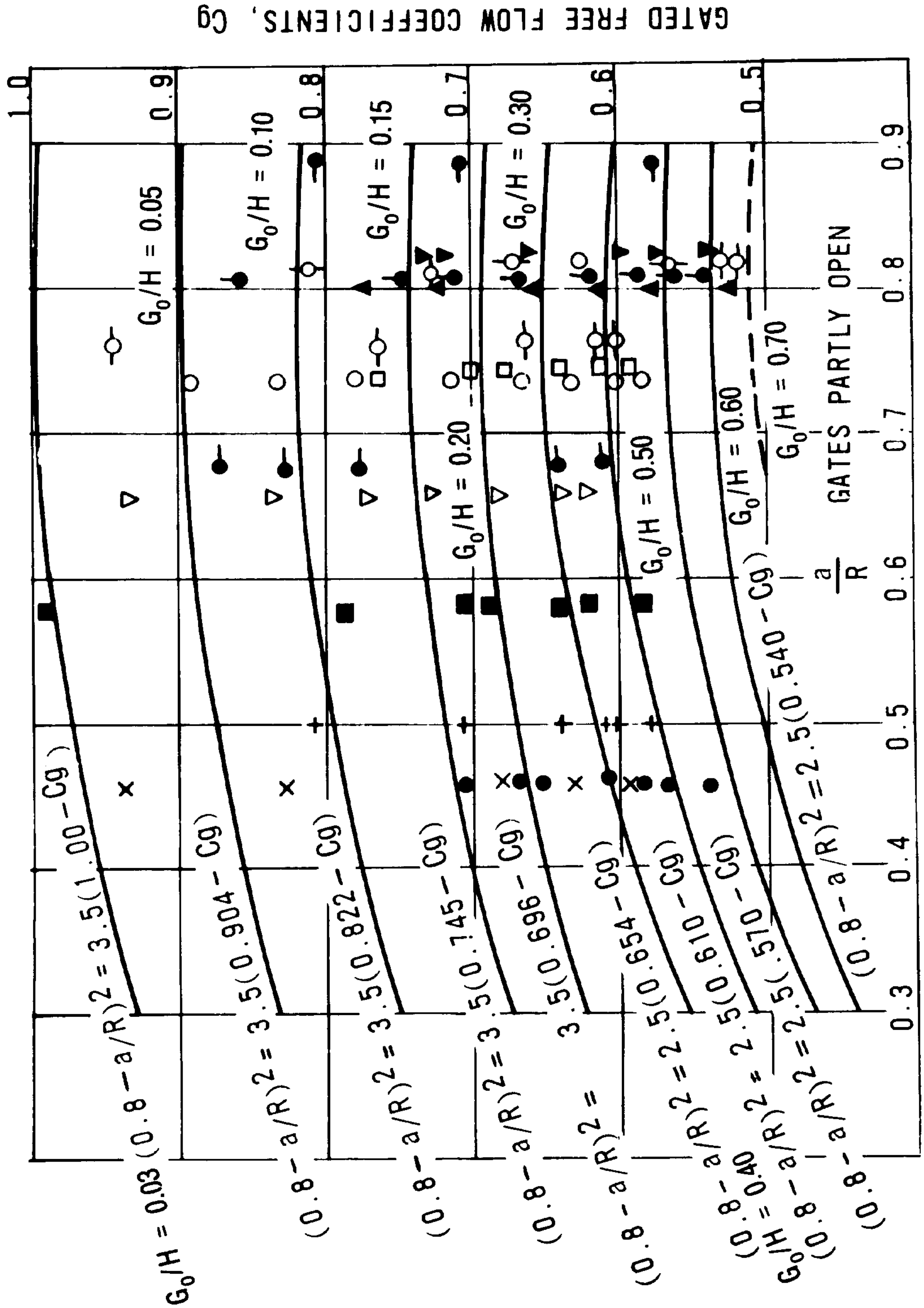
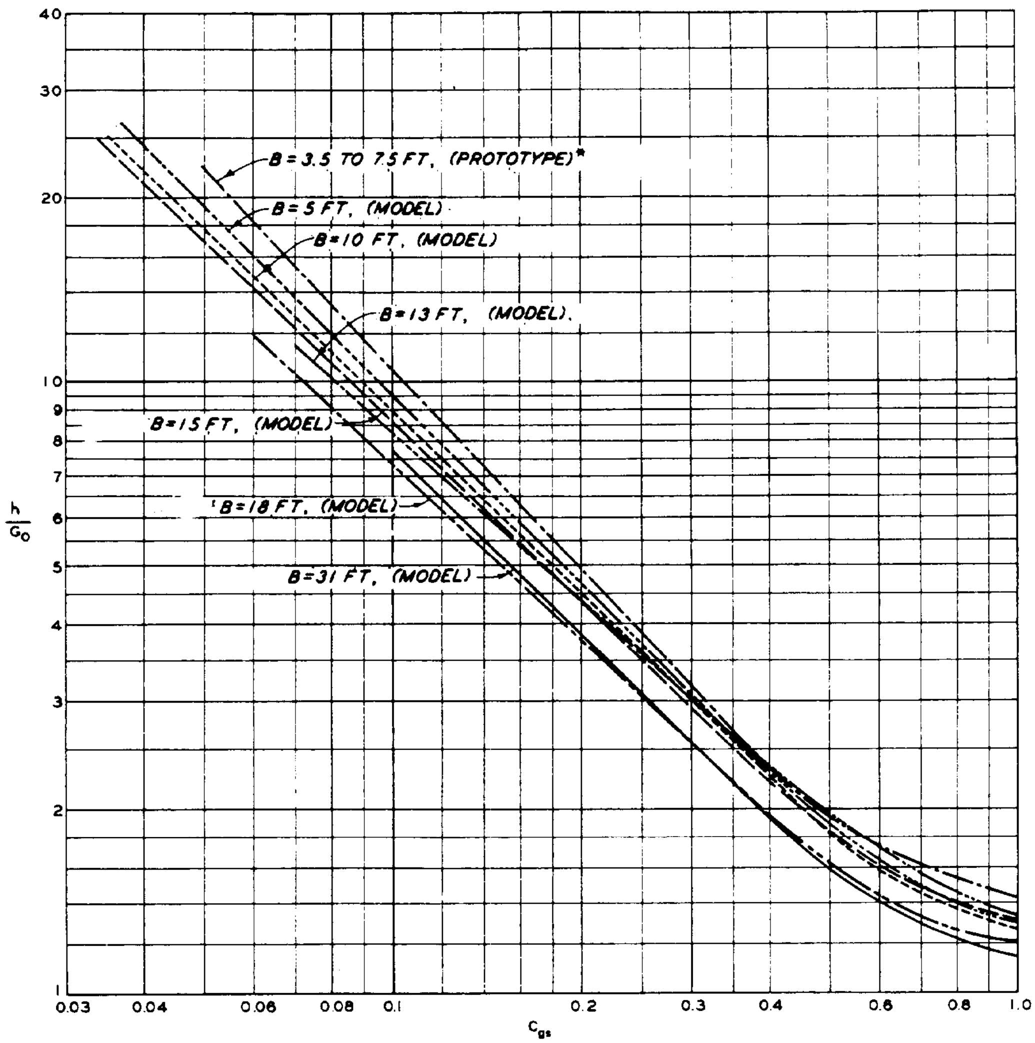


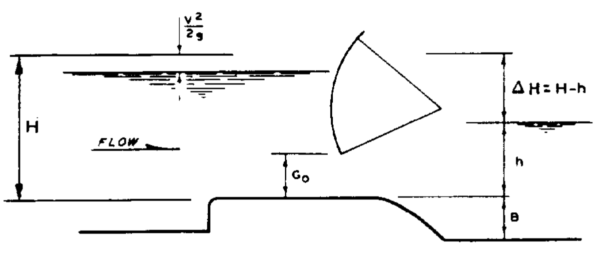
Figure 5-11. Discharge coefficients for free controlled flow
(from item 22, Appendix A)



BASIC EQUATION

$$Q = C_{gs} \; Lh\sqrt{2g\,\Delta H}$$

*MISSISSIPPI RIVER DAMS 2, 5A, AND 26



DEFINITION SKETCH

Figure 5-12. Discharge coefficients for submerged controlled flow (HDC 320-8) rating curve verification. In finalizing rating curves for major navigation systems, special prototype spillway measurements on similar existing projects should be considered.

(2) Tailwater Inaccuracies. Tailwater rating curves are extremely important to the design engineer. The selected tailwater curve will be used in design of spillway capacity, stilling basins, wall heights, foundation drainage, erosion protection, navigation channel depths, and many other critical elements that make up a total project design. It is imperative that the hydraulic engineer have an accurate estimate of what the tailwater curve will be before, during, and after project construction; and throughout the life of the project. The hydraulic engineer must evaluate the likelihood that the tailwater rating will change over this time period and evaluate the extremes to which this change may take place. Furthermore, this information must be passed on to other engineers designing project features so that project integrity will remain as the rating curve shifts. The designer is cautioned against spending too much effort in refining inconsequential parameters, such as spillway pier shape coefficients, without paying sufficient attention to potential shifts in tailwater rating curves which can, of course, have drastic influences on submerged spillway capacity. An example of a very large shift in tailwater rating is shown in Figure 5-13. This figure compares the tailwater ratings for the natural conditions before construction of the Aliceville Lock and Dam on the Tennessee-Tombigbee Waterway with project conditions after construction was complete. The drastic shift of the rating is largely due to excavation of the downstream navigation channel which caused not only an increase in channel flow capacity, but also a significant decrease in channel roughness. The variation in a tailwater rating curve may shift toward more flow capacity, less flow capacity, or oscillate from one to the other and back again. The shift in rating may be abrupt, gradual, or sporadic. It may be caused by sediment erosion or aggradation, excavation or deposition of channel bed or bank material, variations in hydrologic events, loops in rating curves as flow transitions from the rising to falling flood stages, inaccurate estimates of channel roughness, or by man-induced events. The hydraulic engineer should ensure that project features are designed for the proper conditions. For example, for projects with loop rating curves, rising stages should be used for design of stilling basins and erosion protection and falling stages used for setting wall heights. Use of an average tailwater rating curve in this case may yield inadequate design for both wall height and the high-velocity flow areas. The designer might also perform a sensitivity study of various channel "n" values to ensure that an incorrect assumption does not lead to an inadequate design. It will be the primary responsibility of the hydraulic design engineer to recognize the potential for shifts in tailwater ratings, evaluate the magnitude and consequences of a shift, and communicate this knowledge to others on the design team.

5-8. Overflow Embankments.

a. General. Required length of overflow embankments is often determined by selecting the combination of number of gates, length of overflow section, flowage easement, and levee raising that has the least total cost.

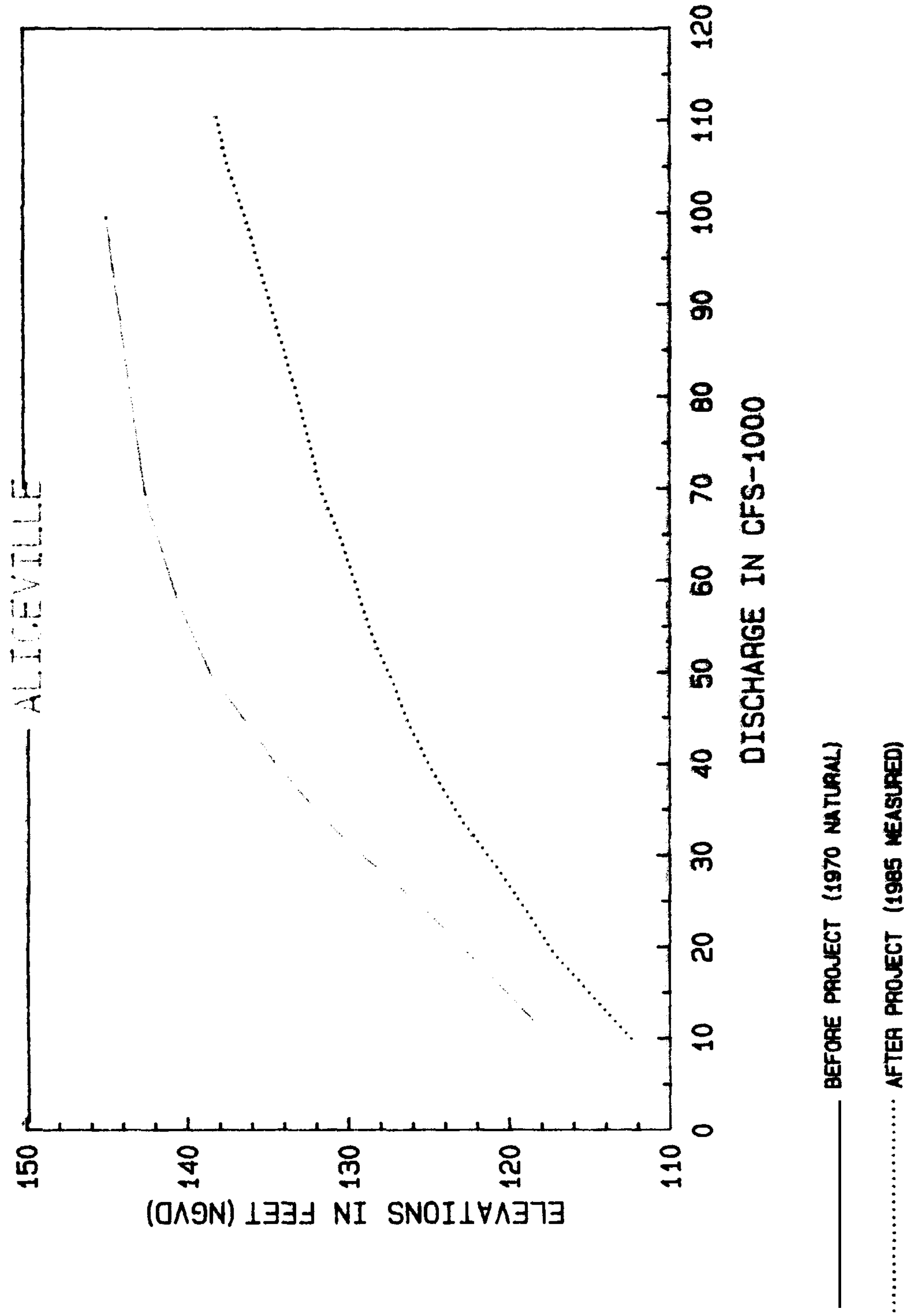


Figure 5-13. Tailwater rating curves, Aliceville Lock and Dam, Tennessee-Tombigbee Waterway

An example of an optimization study accomplishing this is given in Appendix D. When the overflow section operates under only highly submerged conditions the shape of the crest is of little significance on capacity. Overflow sections having significant head differentials will require properly shaped crests (normally ogee), energy dissipation structures, and downstream channel protection. The relatively low embankment sections used on the Arkansas River were designed for submerged conditions with head differentials of up to three feet. These riprap protected embankments are either access or nonaccess embankments having trapezoidal cross sections with a 1V-on-3H upstream face and a 1V-on-4H downstream face. The access embankments have a paved roadway on the crown of the embankment. Detailed discharge and riprap stability guidance is given in item 5 of Appendix A.

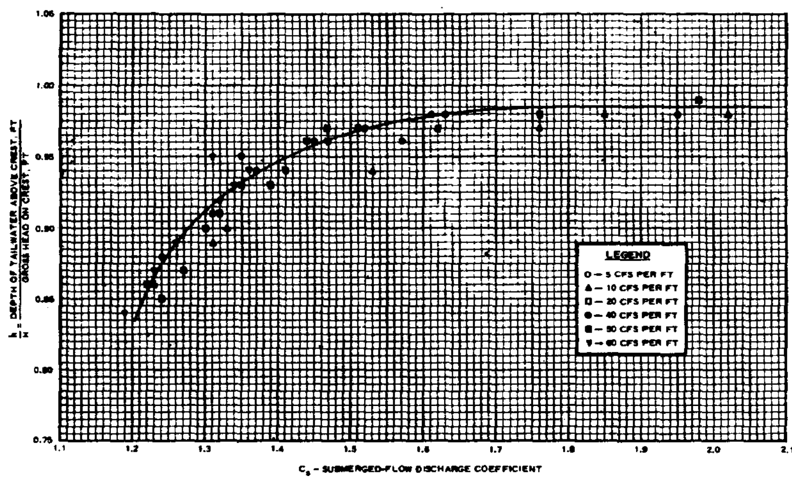
b. Discharge over Uncontrolled Sections. Figure 5-14 shows the submerged flow discharge coefficient for access and nonaccess type embankments. The second type of uncontrolled overflow section is the concrete wall having considerable height and designed to operate under submerged conditions. Discharge coefficients for a rectangular cross section and free flow conditions are shown in Figure 5-15; the reduction in free flow discharge due to submergence is also shown in Figure 5-15.

5-9. Stilling Basin Design.

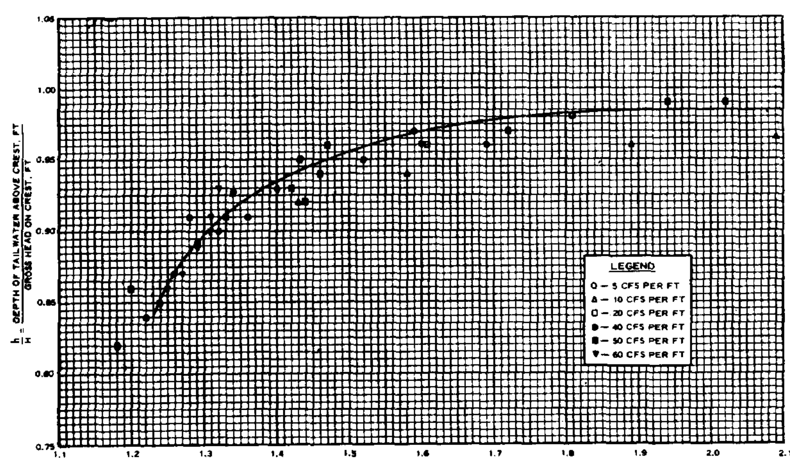
a. General. The purpose of the stilling basin is to reduce the kinetic energy of the flow entering the downstream exit channel. The stilling basin in conjunction with the downstream riprap ensures that local scour downstream of the structure will not undermine or otherwise threaten the integrity of the structure. Model tests can be used to find the optimum combination of stilling basin and downstream channel protection.

b. Influence of Operating Schedules. Operating schedules, both normal and emergency, are vital considerations in stilling basin design. Normal operating schedules should result in approximately equal distribution of flow across the outlet channel. Thus changes in the position of individual gates should be made in small increments with no two gate openings varying more than one foot. However, unusual or emergency operation must be considered. Unusual operation would include passage of floating debris (ice, logs, trash, etc.) through the gated structure during periods of minimum flow in the river. Such debris usually will begin to be drawn under a gate that is about one-third opened (see items 15 and 18, Appendix A). Emergency operation would include design for one gate fully opened during periods of minimum flow which generally means minimum tailwater. Thus these operation requirements dictate a stilling basin that will adequately dissipate the excess kinetic energy at a low tailwater elevation.

c. Requirements for New Project Design. The following three conditions are used to optimize stilling basin length and downstream scour protection thickness, size, and length. Structure foundation should be considered in determining the design condition. Structures founded on rock may have less restrictive energy dissipation and downstream protection requirements.

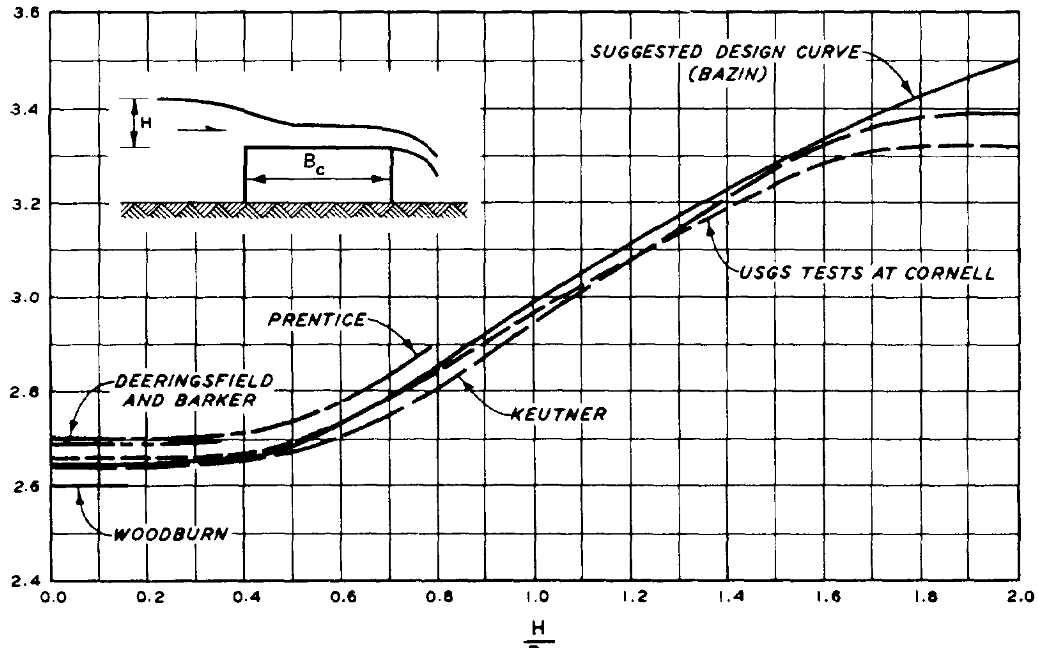


Access type embankments

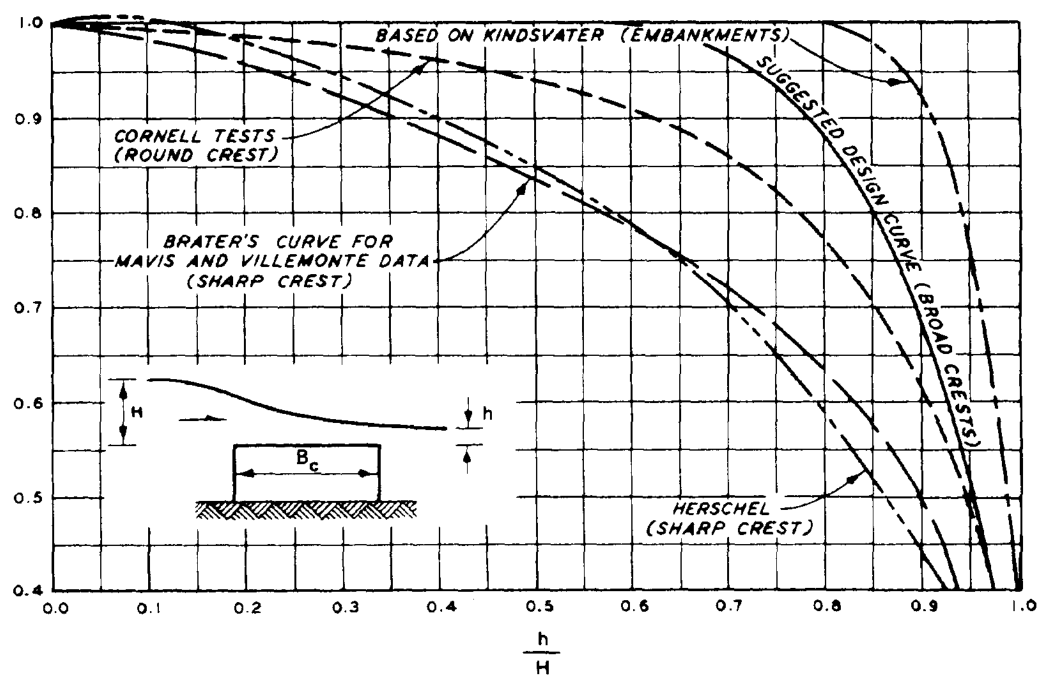


Nonaccess type embankments

Figure 5-14. Discharge coefficients for embankments under submerged flow (from item 5, Appendix A), $Q = C_s Lh \sqrt{2g\Delta H}$



a. FREE FLOW



b. SUBMERGED FLOW

NOTE : C_f = FREE-FLOW COEFFICIENT
C_s = SUBMERGED-FLOW COEFFICIENT
NEGLIGIBLE VELOCITY OF APPROACH

Figure 5-15. Low-monolith diversion, discharge coefficients (from HDC 711)

(1) Uniform discharge through all spillway gates for a range of headwaters and tailwaters expected during project life.

(2) Single gate fully opened with normal headwater and minimum tailwater. This condition would assume gate misoperation or marine accident. Minor damage to the downstream scour protection may occur as long as the integrity of the structure is not jeopardized. Single gate fully opened with above normal pool (perhaps the 50- to 100-year pool) should also be given consideration. This condition would simulate loose barges that could block several gates causing above normal pools as occurred at Arkansas River Lock and Dam No. 2 during December 1982.

(3) Single gate opened sufficiently wide to pass floating ice or drift at normal headwater and minimum tailwater. During preliminary design, a gate half opened can be assured to approximate ice- or drift-passing condition. Final design usually requires model studies to determine the proper gate opening. No damage should occur for this condition. For most low-head navigation structures, conditions (2) and (3) result in free flow over the crest. The stilling basin design guidance presented in this chapter is for free flow, Stilling basins designed for submerged flow normally require a model study.

d. Hydraulics of Stilling Basins. Computations for d_1 and V_1 can be based on the assumption that there is no energy loss between the upper pool and the toe of the jump. The energy equation can be used to determine the entering depth and velocity into the stilling basin according to

$$\text{Upper Pool Elevation} + \text{Velocity Head Upstream} = \text{Stilling Basin Floor Elevation} + \frac{V_1^2}{2g} + d_1 \qquad (5\text{-}8)$$

Knowing the upper pool elevation, velocity head upstream (if significant), and discharge, V_1 and d_1 can be solved by trial and error for an assumed stilling basin floor elevation. Next the Froude number of the flow entering the stilling basin is computed according to

$$F_1 = \frac{V_1}{\sqrt{gd_1}} \qquad (5\text{-}9)$$

Then the momentum equation is used to determine the ratio between the depths before and after the hydraulic jump according to

$$\frac{d_2}{d_1} = 0.5\left(\sqrt{1 + 8F_1^2} - 1\right) \qquad (5\text{-}10)$$

(This form of the momentum equation ignores the forces on baffle blocks in the analysis. A comprehensive treatment of these forces in the momentum equation is given in item 2 of Appendix A.) At this point, the assumed stilling basin elevation is checked against the available tailwater according to

$$\text{Tailwater for Given Discharge} - \text{Assumed Stilling Basin Floor Elevation} = \text{Factor } (d_2) \qquad (5\text{-}11)$$

A new stilling basin floor elevation is assumed until Equation 5-11 is satisfied. Early stilling basin design guidance used a factor equal to 1.0. Recent guidance has allowed higher stilling basin floor elevations by setting this factor equal to 0.85 when used with baffle blocks and an end sill. The higher stilling basin floor elevation often improves performance at intermediate discharges and results in lower cost. Use of a factor less than 1.0 in Equation 5-11 can only be used in conjunction with Equation 5-10, the simplified momentum approach.

e. Recommendations from Results of Previous Model Tests.

(1) General. Model tests have been conducted at WES, Vicksburg, Miss. (items 10, 13-16 of Appendix A), during which stilling basin designs were developed for one gate half or fully opened. Recommendations from results of these tests are summarized in Table 5-1 and in the paragraphs that follow. The energy dissipators for one gate half or fully opened are not hydraulic-jump type stilling basins. These basins often have entering Froude numbers less than 4.0 which means they are inefficient and unstable--the flow will oscillate between the bottom and water surface resulting in irregular wave formation propagating downstream. Baffles and end sills help to stabilize low Froude number basins. Primary dissipation results from impact of the jet against the baffles, which also assists lateral spreading of the jet, with tailwater as a supporting element. In a hydraulic-jump type stilling basin, tailwater is a primary force and baffles are supporting elements ; lateral spreading of the jet, outside of the confining walls, usually is not a consideration.

(2) Basin Elevation. In a baffle-assisted hydraulic-jump type stilling basin, the apron must be placed at an elevation that allows tailwater to provide a depth on the apron of at least $0.85d_2$ (factor = 0.85). In the stilling basin considered herein, this has not proved to be a rigid requirement. However, for initial design of a specific project and until it has been established in model tests that conditions at that project will permit an apron at a higher elevation, it is suggested that the apron be placed at an elevation that will provide a tailwater depth of at least $0.85d_2$ for both one gate half or fully opened.

(3) Basin Length. Items 10 and 13-16 of Appendix A suggest a required length, L_2 from toe of jump to beginning of 1V-on-5H upslope of

$$L_2 = 2d_1F_1^{1.5} \qquad (5\text{-}12)$$

TABLE 5-1
Hydraulic Design Data from Physical Model Studies for Low-Head Navigation Dam Spillways
Based on Single-Gate Opening (Fully and/or Half) Criteria

Project Name	Item No.	Basin No.	Designed for What Gate Opening	Unit Discharge cfs/ft	Bay Width ft	Entering Froude No. F'*	d_2 ft	$\frac{TW}{d_2}$	$\frac{L_1}{d_2}$	Baffle Height / d_2	$\frac{L_2}{d_2}$	End Sill Height / d_2	d_{50}† ft
L&D 26	15	16	Full	775	110	2.5	44.5	0.81	1.08	0.27	2.6	0.25	3.8
Aliceville	13	6	Full	350	60	3.5	30.5	0.72	1.31	0.26	2.6	0.16	2.2
Columbus	16	5	Full	350	60	3.7	31.0	0.77	1.29	0.26	2.6	0.16	2.0
Red River No. 1	14	16	Full	484	50	3.9	39.6	0.81	1.26	0.25	2.8	0.43	1.7
Red River No. 2	10	13	Full	683	60	2.4	39.5	0.71	1.06	0.23	2.5	0.18	2.5
Red River No. 3	**	2	Full	817	60	2.75	47.9	0.79	1.15	0.21	3.0	0.15	2.6
L&D 26	15	30	Ice and Debris	382	110	3.7	32.8	0.91	1.58	0.24	3.0	0.15	2.6
Columbus	16	4	Half	242	60	4.4	26.2	0.84	1.53	0.31	3.1	0.19	1.7
Red River No. 1	14	9	Half	390	50	3.6	33.2	0.72	1.51	0.33	3.0	0.09	
Red River No. 1	14	17	Half	370	50	3.7	32.3	0.74	1.55	0.32	3.1	0.09	1.5
Red River No. 1	14	7	Half	370	50	4.1	33.7	0.71	1.48	0.34	2.7	0.27	1.1

* Assumes no energy loss between upper pool and stilling basin.
** Unpublished draft report.
† d_{50} size immediately downstream from end sill.
Definition sketch shown in Figure 5-1.

(4) Baffles. The position and height of the first row of baffles have a major influence on stilling action. Baffle height and position recommended for the basins developed in items 10 and 13-16 of Appendix A are as follows:

Gate Opening	Height h_b	Distance to First Row L_1
Full	$0.25d_2$	$1.3d_2$
Half	$0.3d_2$	$1.5d_2$

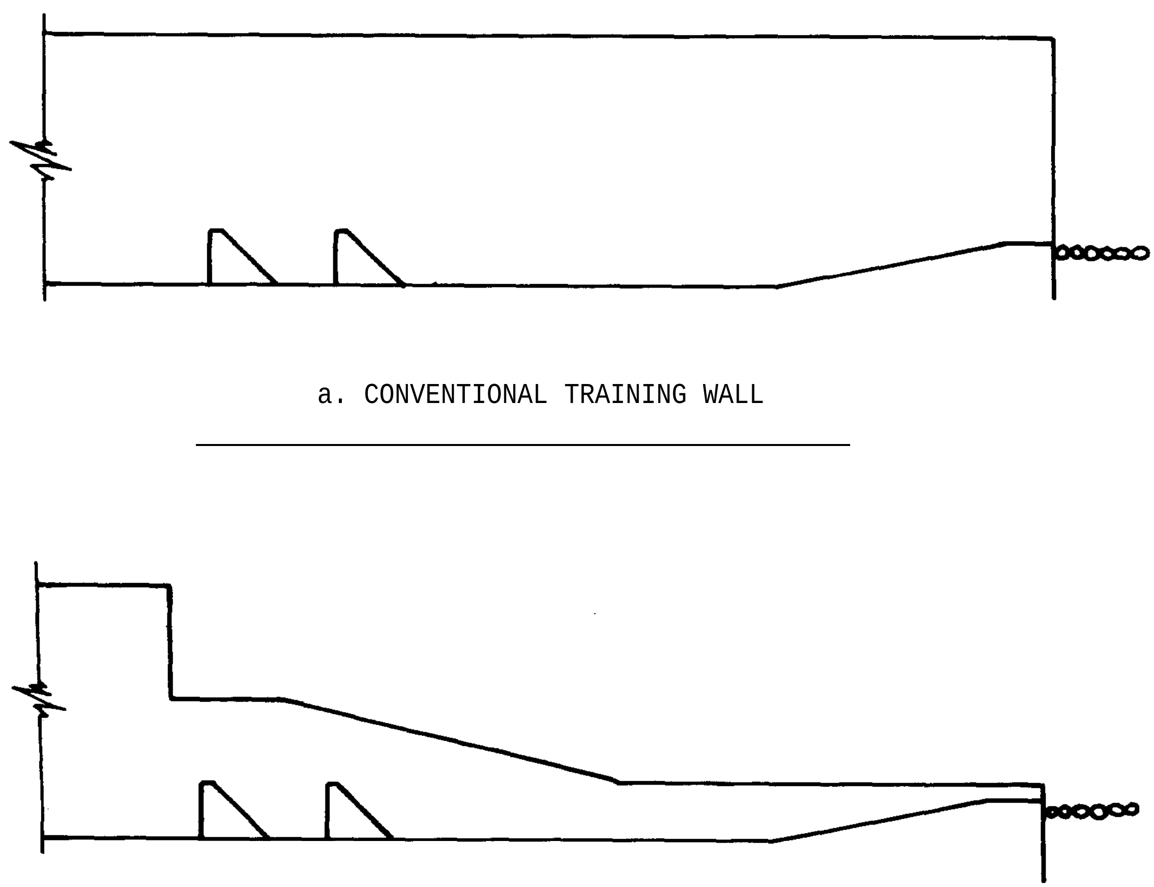
These basins designed for a single gate half or fully opened require higher baffle blocks than hydraulic-jump type basins. A second row of baffles is not required for maintaining the jump within the basin but is recommended to reduce attack on the downstream channel protection. These baffles should be the same height as those in the first row, placed with their upstream faces about two baffle heights downstream from the upstream faces of the first row and staggered with respect to the baffles in the first row. Reference item 2 of Appendix A for determining forces on baffle blocks. In cases where foundation requirements dictate a deep basin ($>d_2$), baffle blocks may not be required.

(5) Gate Pier Extensions. Gate pier extensions are required to extend into the basin to a position five feet upstream of the baffles to prevent return flow from inoperative bays. The pier extension can be extended farther downstream if required for stability. These extensions are required to ensure adequate stilling basin performance for the single gate half- and fully opened criteria given in paragraphs 5-9c(2) and 5-9c(3), respectively. The pier extensions should be at least one foot higher than the tailwater used for the single gate half- or fully opened criteria. Pier extension width can be less than the main spillway piers.

(6) End Sill. An end sill slope of 1V on 5H was effective in spreading the flow for single gate operation. The higher the end sill, the more effective it will be in spreading the jet during single gate operation, but there are limitations. The higher end sill results in shallower depths in the exit channel and possibly higher velocities over the riprap. Of course, the top of the end sill should not be appreciably above the exit channel. Also, the end sill should not be so high that it causes flow to drop through critical depth and form a secondary jump downstream. To prevent this, the Froude number $F = V/\sqrt{gd}$ at the top of the end sill, calculated as described below, should not exceed 0.86 for single gate guidance given in paragraph 5-9c. In this calculation, V is difficult to determine because of spreading of the flow for single gate operation. A reasonable estimate for V is 80 percent of the velocity over the end sill without spreading based on bay width, discharge, and depth over end sill. The terms d and g represent depth of tailwater over the end sill and the acceleration due to gravity, respectively. Experiments in a rectangular channel indicated that tranquil flow becomes unstable when F exceeds 0.86; thus this limiting value. Excessive spreading will cause attack of boundaries in outside bays. Based on items 10, and 13-16 of Appendix A, the end-sill height varied considerably for basins designed for either fully or half-opened gate criteria. A value of

0.15 to $0.20d_2$ is recommended for basins designed for either a fully or half-opened gate.

(7) Training Walls. The elevation of the top of the training walls is normally selected to prevent overtopping at all but the highest discharges. This is not a strict requirement for low-head navigation dams and training wall tops have been placed as low as two feet above the downstream normal pool elevation. This reduction in height should be model tested. Training walls are normally extended at a constant top elevation to the end of the stilling basin as shown in Figure 5-16a. This, too, is not a strict requirement. The Red River design is shown in Figure 5-16b. Adjacent project features and topography have a significant impact on training wall design. Reference EM 1110-2-1603 for determining hydraulic forces (static and dynamic) on stilling basin training walls.



a. CONVENTIONAL TRAINING WALL

b. RED RIVER # 3 TRAINING WALL

Figure 5-16. Training walls

(8) Abrasion. Abrasion of concrete can be caused by the presence of gravel or other hard particles. Rock, gravel, scrap metal, and other hard material may find their way into the energy dissipator by various means. Rock

may be carried into a stilling basin over the top of low monoliths during construction, by rollers or eddies bringing debris in from downstream, or by cobbles moving as bed load. Protection stone in the vicinity of the end sill should not contain stone sizes that can be transported by underrollers into the stilling basin. In some cases, the contractor may fail to clean out all hard, loose material after construction. During operation, rocks may be thrown in from the sidewall by the public, or fishermen using rocks for anchors may leave them behind. The elimination of such material may require specification of construction practices or proper restriction of the public during operation. In cases where it is believed that rock and gravel are being transported into the basin by rollers, all gates should discharge an equal amount of water.

(9) Cavitation is the successive formation and collapse of vapor pockets in low-pressure areas associated with high-velocity flow. Cavitation damage can occur on the sides of baffle blocks, on the floor of a stilling basin just downstream from baffle blocks, and at construction joints near the upstream end of the stilling basin. Any surface discontinuity of the boundary into or away from high-velocity flow can cause cavitation. Relative movement of two concrete monoliths or slabs with a lateral construction joint so that the downstream slab comes to rest higher than the upstream slab produces a situation where cavitation may result. In any case where high-velocity flow tends to separate from the solid boundary, cavitation may be expected to exist. Cavitation is not normally a problem at low-head navigation dams because of the relatively low velocities. There is reason to believe that both abrasion and cavitation are responsible for damage at some structures. If a sizable depression in the concrete surface is eroded by abrasion, cavitation may then form and augment the damage. Likewise abrasion can mask cavitation where both are occurring. In general, concrete damaged by cavitation has a ragged angular appearance as though material had been broken out of the mass. In contrast, damage caused by abrasion has a smoother or rounded appearance, such as would be caused by grinding. Reference EM 1110-2-1602 for additional guidance relative to cavitation.

5-10. Approach Area.

a. Configuration. The approach to the spillway should be greater than three feet below the crest of the spillway. An approach depth of five feet is recommended because most discharge calibration data were taken with this depth. Approaches with depths less than three feet can result in greater tendency for movement of the riprap in front of the structure for a single gate fully opened. Approaches having a deep tranch in front of the structure can result in instabilities of the flow over the crest and may simply fill with sediment. The approach should be horizontal for a minimum of 50 feet and then sloped to the streambed at a rate not to exceed 1V on 20H.

b. Upstream Channel Protection. To prevent scour upstream of the structure, protection is required, particularly for single gate operation. An estimate of the required riprap size upstream of a navigation dam can be obtained by determining the approach velocity by taking the unit discharge (discharge/width of bay) and dividing by the depth (difference in elevation between the upper pool and the approach channel to the spillway). This provides an average velocity and depth that can be used in the following relation to determine the stone size required.

$$\frac{D_{\%}}{\text{depth}} = C\left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w}\right)^{1/2} \frac{V}{\sqrt{g\ \text{depth}}}\right]^{2.5} \qquad (5\text{-}13)$$

The following coefficients are recommended for riprap design in low turbulence open channel flow:

$D_{\%}$	Safe Design, C	Gradation	Thickness
D_{50} (Min)	0.44	Table 5-2	1.0 D_{100}(MAX)
D_{50} (Min)	0.30	Table 5-3	1.5 D_{100}(MAX)
D_{30} (Min)	0.375	d_{85}/d_{15} = 1.35-4.6	1.0 D_{100}(MAX)

The safe design C is equal to 1.25 times the C determined for incipient failure. See item 11 for additional information. Placement underwater requires an increase in thickness of 50 percent. Single gate operation will generally be the most severe with respect to design of upstream riprap but hinged pool operation (as described in paragraph 7-3(c)) should be evaluated. Concrete aprons have been used in place of riprap when riprap size becomes excessive. The riprap or concrete apron should be extended upstream a minimum distance equal to the head on the crest. If protection must be provided for the effects of sunken barges in front of the structure, the concrete apron should be used.

5-11. Exit Area.

a. Configuration. For the condition of only a single gate discharging, configuration of the exit area has a major influence on stilling action. Abrupt side contractions and areas of unequal elevation across the channel cause side eddies to be intensified and thus hamper jet spreading. There is little agreement on the effectiveness of a preformed scour hole. Many projects have been designed with a deepened area downstream to lessen attack on the riprap. A relatively small amount of expansion, preferably both vertically and horizontally, will reduce the severity of attack of the channel boundary. However, there is a tendency for this deepened exit channel to exhibit stronger side eddies which tends to reduce spreading for single gate operation and can lead to a decrease in riprap stability. Final riprap configurations downstream from spillways should be model-tested and adjusted as necessary to ensure the adequacy of the protection. Based on the above field and model experiences the following guides for preliminary layout are suggested. Begin the riprap with the top of the blanket 1 to 2 feet below the top of the basin end sill, If possible, extend the riprap section horizontally. Where the streambed is higher than the end sill, slope the riprap upward on a 1V-on-20H slope. Where locks or other structures do not abut the spillway the riprap section is extended up the bank-line slope. The toe of this slope should be set back 5 to 10 feet from the face of the spillway training wall. These guides are illustrated in Plates 5-4 to 5-6 (example at end of this chapter).

b. Downstream Channel Protection. The size and extent of the riprap required in the exit area depend upon the effectiveness of the stilling basin, tailwater depth in the exit, and configuration of the exit area. The size of riprap required is almost always governed by either the fully or half-opened gate criteria or diversion conditions. As flow leaves the single gate bay, spreading occurs and the average velocity decreases in the downstream direction. At the end sill the average velocity over the end sill can be 75 to 90 percent of the velocity without spreading. Results from items 10 and 13-16 of Appendix A show a wide variation in required riprap size. Use of 80 percent of the velocity over the riprap without spreading in the relation

$$V = 1.12 \left[2g \left(\frac{\gamma_s - \gamma_w}{\gamma_w} \right) \right]^{1/2} D_{50}(\mathrm{MIN})^{1/2} \qquad (5\text{-}14)$$

provides riprap size for use immediately downstream of the end sill. This equation is restricted to basins designed using the guidance presented in this chapter. This equation is the same form as the Isbash relation given in HDC- 712-1. A comparison of the results given in Table 5-1 and Equation 5-14 is given in the following:

Project Name	Basin No.	Velocity over Riprap Without Spreading, ft/sec	D_{50} Model, feet	D_{50} Computed, feet
L&D 26	16	29.9	3.8	4.3
Aliceville	6	19.4	2.2	1.8
Columbus	5	17.6	2.0	1.5
RR 1	16	30.2	1.5	4.4
RR 2	13	31.1	2.5	4.7
RR 3	2	25.8	2.6	3.2
L&D 26	30	14.7	2.6	1.0
Columbus	4	13.4	1.7	0.9
RR 1	17	16.8	1.5	1.4
RR 2	7	23.4	1.1	2.7

The large differences between model and computed results are largely due to difference in stilling basin performance, particularly the effects of a wide variation in end-sill height. These values should be used in preliminary design and verified in a physical model. Riprap gradations are given in Table 5-3 for placement in the dry. Thickness for placement in the dry should be $1.5D_{100}$(MAX) or $2.0D_{50}$(MAX), whichever is greater. Thickness for placement underwater should be increased 50 percent. The top of the riprap should be placed one to two feet below the top of the end sill. Total length of riprap protection on the channel invert downstream of the end sill ranged from $4d_2$ to

$27d_2$ in items 10 and 13-16 of Appendix A. A minimum length of $10d_2$ downstream of the end sill is recommended for fully or half-opened gate design. The change in riprap size in the downstream direction should be as follows:

Distance	Riprap Size
$3d_2$	x = thickness immediately downstream of end sill
Next $3d_2$	0.8x
Next $2d_2$	0.6x
Next $2d_2$	0.4x

TABLE 5-2

Gradations for Riprap Placement in the Dry, Low Turbulence Zones

Percent Lighter by Weight	Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds	
Specific Weight = 155 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	81	32	159	63	274	110	435	174
50	24	16	47	32	81	55	129	87
15	12	5	23	10	41	17	64	27
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	649	260	924	370	1,268	507	1,688	675
50	192	130	274	185	376	254	500	338
15	96	41	137	58	188	79	250	105
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	2,191	877	3,480	1,392	5,194	2,078	7,396	2,958
50	649	438	1,031	696	1,539	1,039	2,191	1,479
15	325	137	516	217	769	325	1,096	462

(Continued)

TABLE 5-2 (Concluded)

Percent Lighter by Weight	Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds	
Specific Weight = 165 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	86	35	169	67	292	117	463	185
50	26	17	50	34	86	58	137	93
15	13	5	25	11	43	18	69	29
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	691	276	984	394	1,350	540	1,797	719
50	205	138	292	197	400	270	532	359
15	102	43	146	62	200	84	266	112
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	2,331	933	3,704	1,482	5,529	2,212	7,873	3,149
50	691	467	1,098	741	1,638	1,106	2,335	1,575
15	346	146	549	232	819	346	1,168	492
Specific Weight = 175 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	92	37	179	72	309	124	491	196
50	27	18	53	36	92	62	146	98
15	14	5	27	11	46	19	73	31
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	733	293	1,044	417	1,432	573	1,906	762
50	217	147	309	209	424	286	565	381
15	109	46	155	65	212	89	282	119
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	2,474	990	3,929	1,571	5,864	2,346	8,350	3,340
50	733	495	1,164	786	1,738	1,173	2,474	1,670
15	367	155	582	246	869	367	1,237	522

TABLE 5-3

Gradations for Riprap Placement in the Dry, High Turbulence Zones

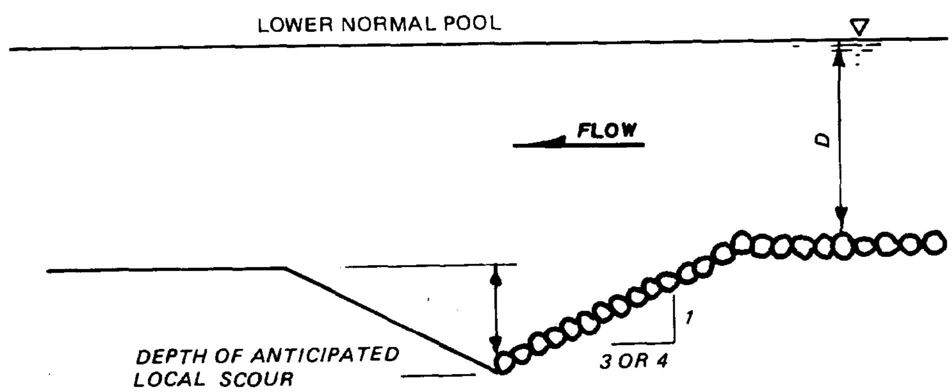
Percent Lighter by Weight	Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds	
Specific Weight = 155 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	24	10	47	19	81	32	129	52
50	7	5	14	9	24	16	38	26
15	4	2	7	3	12	5	19	8
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	192	77	274	110	376	150	500	200
50	57	38	81	55	111	75	148	100
15	28	12	41	17	56	23	74	31
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	649	260	1,031	412	1,539	616	2,191	877
50	192	130	305	206	456	308	649	438
15	96	41	153	64	228	96	325	137
Thickness =	60 Inches		66 Inches		72 Inches		78 Inches	
100	3,006	1,202	4,001	1,600	5,194	2,078	6,604	2,642
50	890	601	1,185	800	1,539	1,039	1,957	1,321
15	445	188	593	250	770	325	978	413
Thickness =	84 Inches		90 Inches		96 Inches		102 Inches	
100	8,248	3,299	10,145	4,058	12,312	4,925	14,768	5,907
50	2,444	1,650	3,006	2,029	3,648	2,462	4,376	2,954
15	1,222	516	1,503	634	1,824	770	2,188	923
Specific Weight = 165 lb/cu ft								
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	26	10	50	20	86	35	137	55
50	11	5	21	10	36	17	58	27
15	5	2	11	3	18	5	29	9
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	205	82	292	117	400	160	532	213
50	86	41	123	58	169	80	225	106
15	43	13	62	18	84	25	112	33

(Continued)

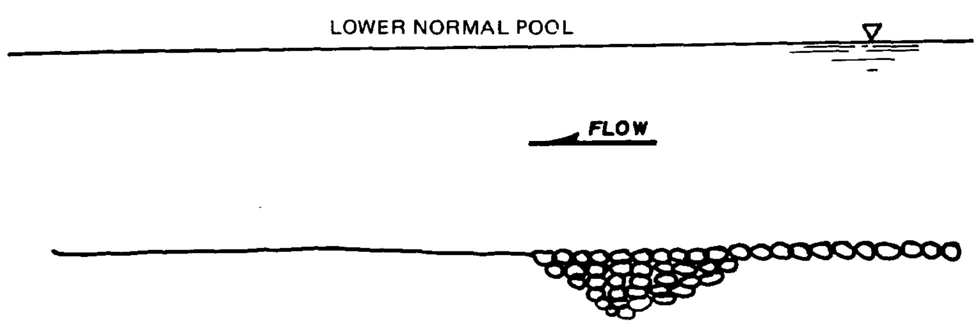
TABLE 5-3 (Concluded)

Percent Lighter by Weight	Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds		Limits of Stone Weight, pounds	
	Specific Weight = 165 lb/cu ft (continued)							
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	691	276	1,098	439	1,638	655	2,333	933
50	292	138	463	220	691	328	984	467
15	146	43	232	69	346	102	492	146
Thickness =	60 Inches		66 Inches		72 Inches		78 Inches	
100	3,200	1,280	4,259	1,704	5,529	2,212	7,030	2,812
50	948	640	1,262	852	1,638	1,106	2,083	1,406
15	474	200	631	266	819	346	1,041	439
Thickness =	84 Inches		90 Inches		96 Inches		102 Inches	
100	8,780	3,512	10,799	4,320	13,106	5,243	15,720	6,288
50	2,602	1,756	3,200	2,160	3,883	2,621	4,658	3,144
15	1,301	549	1,600	675	1,942	819	2,329	983
	Specific Weight = 175 lb/cu ft							
Thickness =	12 Inches		15 Inches		18 Inches		21 Inches	
100	27	11	53	21	92	37	146	58
50	11	5	22	11	39	18	61	29
15	6	2	11	3	19	6	31	9
Thickness =	24 Inches		27 Inches		30 Inches		33 Inches	
100	217	87	309	124	424	170	536	226
50	92	43	130	62	179	85	238	113
15	46	14	65	19	89	27	119	35
Thickness =	36 Inches		42 Inches		48 Inches		54 Inches	
100	733	293	1,164	466	1,738	695	2,474	990
50	309	147	491	233	733	348	1,044	495
15	155	46	246	73	367	109	522	155
Thickness =	60 Inches		66 Inches		72 Inches		78 Inches	
100	3,394	1,357	4,517	1,807	5,864	2,346	7,456	2,982
50	1,006	679	1,338	903	1,738	1,173	2,204	l,491
15	503	212	669	282	869	367	1,105	466
Thickness =	84 Inches		90 Inches		96 Inches		102 Inches	
100	9,312	3,725	11,454	4,581	13,901	5,560	16,673	6,669
50	2,759	1,862	3,394	2,291	4,119	2,780	4,940	3,335
15	1,380	582	1,697	716	2,059	869	2,470	1,042

Riprap creates locally high boundary turbulence that leads to local scour at the downstream end of the riprap blanket. This requires that the downstream end of the riprap be "keyed in" as shown in Figure 5-17. Method A requires extending the riprap to a depth equal to or greater than the anticipated scour. Method B provides sufficient riprap in a trench to launch as local scour occurs, EM 1110-2-1601 provides guidance for designing riprap end protection. The need to "key in" the riprap is most apparent at projects where the downstream riprap protection does not extend $10d_2$ below the end sill. In some cases, adjacent vertical walls inhibit spreading of the jet during single gate operation and increase the size of riprap required. In cases where the riprap size becomes excessive, concrete aprons or grout-filled bags have been used. Side-slope riprap is normally the same size as the invert. If required, riprap downstream of the $10d_2$ limit should be designed according to EM 1110-2-1601. Granular filters are recommended for riprap placement adjacent to structures. EM 1110-2-1901 presents guidance for filter design.



a. METHOD A - EXTEND TO ANTICIPATED SCOUR DEPTH



b. METHOD B - PROVIDE ROCK-FILLED TRENCH

Figure 5-17. Methods for transitioning from riprap to the unprotected downstream channel

5-12. Spillway Gates. Various types of gates have been used as control devices at Corps of Engineers navigation projects. Examples are tainter gates, roller gates, vertical-lift gates, etc. The current most commonly used and recommended control is the tainter gate.

5-13. Gate Types and Selection. The types of gates used at Corps of Engineers navigation dams and factors considered in the selection of type of gate at a specific project are described in the following paragraphs.

a. Roller Gates. A roller gate is a long metal cylinder with "ring gears" at each end that mesh with inclined metal racks supported by the piers. The cylinder is braced internally to act as a beam to transmit the water load into the piers. The effective damming height of the structural cylinder can be increased by means of a projecting apron that rotates into contact with the sill as the gate rolls down the inclined racks. The gate is raised and lowered by means of a chain wrapped around one end of the cylinder and operated by a hoist permanently mounted in the pier. The rolling movement of the gate and the limited amount of frictional contact at the sealing points permit comparatively fast operation with a small expenditure of power. Roller gates have been built with a damming height of 30 feet, with lengths up to 125 feet on pile foundations and 150 feet on rock foundations.

b. Tainter Gates. A tainter gate in its simplest form is a segment of a cylinder mounted on radial arms that rotate on trunnions embedded in the piers. The tainter gate is considered the most economical, and usually the most suitable, type of gate for controlled spillways because of its simplicity, light weight, and low hoist-capacity requirements. The use of side seals eliminates the need for gate slots that are conducive to local low-pressure areas and possible cavitation damage. The damming surface consists of a skin plate and a series of beams that transmit the water load into the radial supporting arms. The tainter gate is raised and lowered by chains or wire rope attached at both ends, since the tainter type is less capable of resisting torsional stress than the roller gate. Gates may be manipulated by a traveling hoist, or by individual hoists, depending upon the desired speed of operation and consideration of costs. Tainter gates require more power for operation than roller gates of similar size, since nearly all the weight of the gate is suspended from the hoisting chains while the weight of a roller gate is about equally divided between the chain and the pier. Counterweights will reduce power required, but will add to the total weight of the structure. Tainter gates built to heights of 75 feet and lengths of 110 feet have been used for navigation dams. It is desirable but not mandatory that the trunnions of tainter gates be placed above high water, and essential that the gate itself be capable of being raised above high water. Item 3 of Appendix A identifies three types of tainter gate mounting arrangements and describes, with pertinent geometrical data, the gate design and mounting arrangement at 176 Corps of Engineers projects.

c. Vertical-Lift Gates. The vertical-lift gate moves vertically in slots formed in the piers and consists of a skin plate and horizontal girders that transmit the water load into the piers. For the larger heads, the gate must be mounted on rollers to permit movement under water load. The vertical-lift gate, like the tainter gate, must be hoisted at both ends, and the entire weight is suspended from the hoisting chains. Piers must be extended to a considerable height above high water in order to provide guide slots for the gate in the fully raised position. Vertical-lift gates have been designed for spans in excess of 100 feet. High vertical-lift gates are sometimes split into two or more sections in order to reduce hoist capacity, reduce damage to fingerlings passing downstream, or ease passing ice and debris. However, this does increase operating difficulties, because the top leaf or leaves have to be removed and placed in another gate slot.

d. Other Types. Various other types of damming surfaces have been used for navigation dams. These usually have been relatively slow-acting adaptations of stop-log bulkheads or needle dams for operation by hand or limited amounts of mechanical power. The stop-log type of dam consists of piers with vertical slots in which timbers or built-up sections of skin plate and girders are stacked to the desired height. The needle dam consists of a sill and piers that support a girder designed for horizontal loading. Needles or shutters of comparatively narrow width are placed vertically or inclined downstream to rest against the girder and sill and are held in place by the water load. Other navigation dam types such as wicket (Chanoine and Bebout), bear trap, and Boule’dam (see Figure 5-18) are movable dams that are no longer being constructed but are still being used.

e. Selection of Gates. Gates that best meet the operational requirements of the proposed spillway should be provided. Where two or more types of gates appear equally efficient, from a functional standpoint, the decision should be made upon an economic basis. Tainter gates have been used in most recently constructed navigation dams. The following advantages may be ascribed to tainter gate installations:

(1) Lighter lifting weight with smaller hoist requirements.

(2) Adaptable to fixed individual hoists and push-button operation. Individual hoists may have a lower first cost than gantry cranes and require fewer operating personnel.

(3) Less time required for gate operation (more than one gate can be operated at the same time.

(4) Favorable discharge characteristics.

Disadvantages of tainter gate installations are:

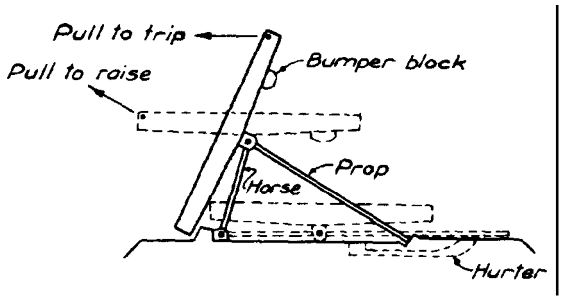
(1) Encroachment of radial arm on the water passage.

(2) The necessity for excessively long radial arms where the flood level, to be cleared, is extremely high.

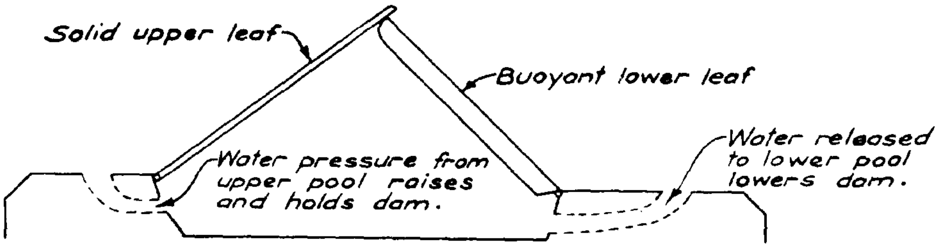
The advantages of a vertical-lift gate installation are:

(1) Provision of a clear gate opening with no encroachment, when raised, of any part of the gate structure on the water passage.

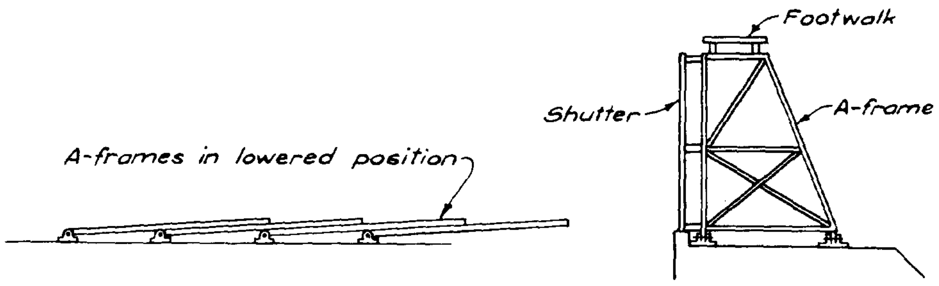
(2) More adaptable to extreme pool fluctuations in that it is lifted bodily out of the water.



WOODEN CHANOINE WICKET

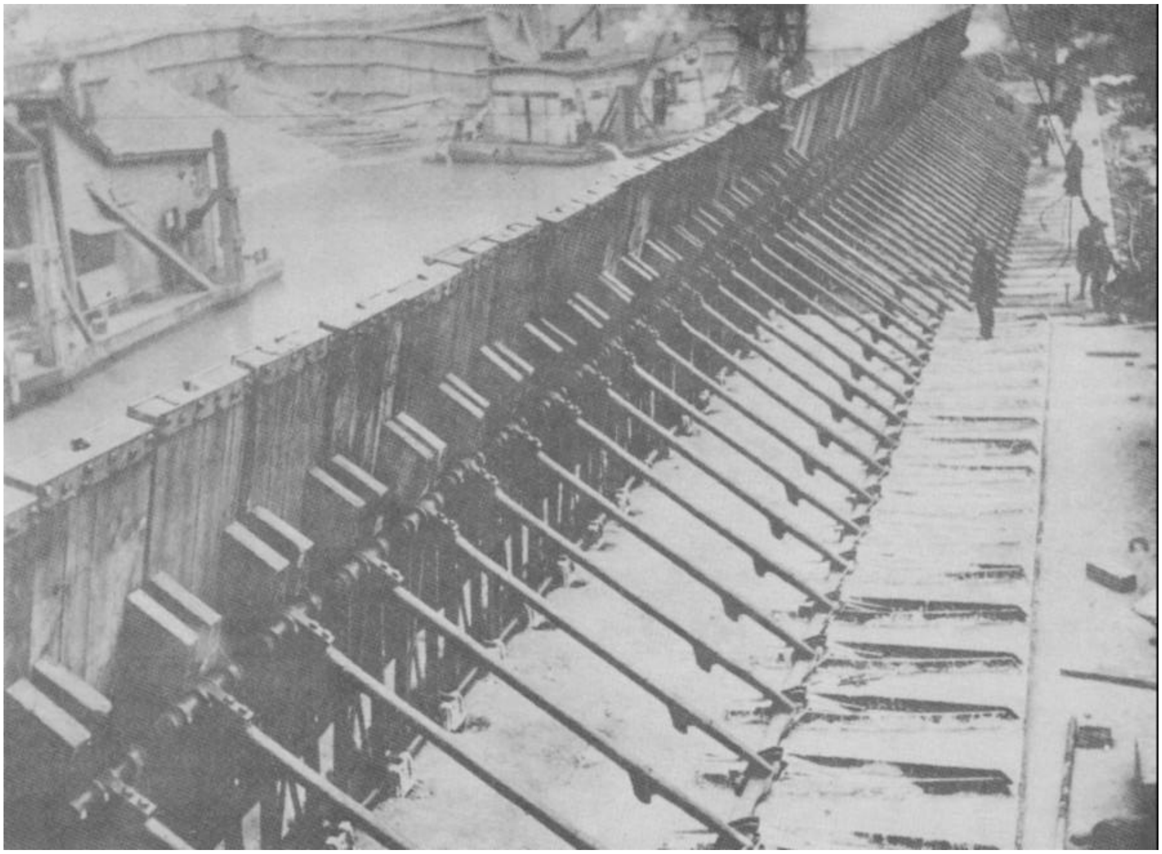


BEAR TRAP DAM

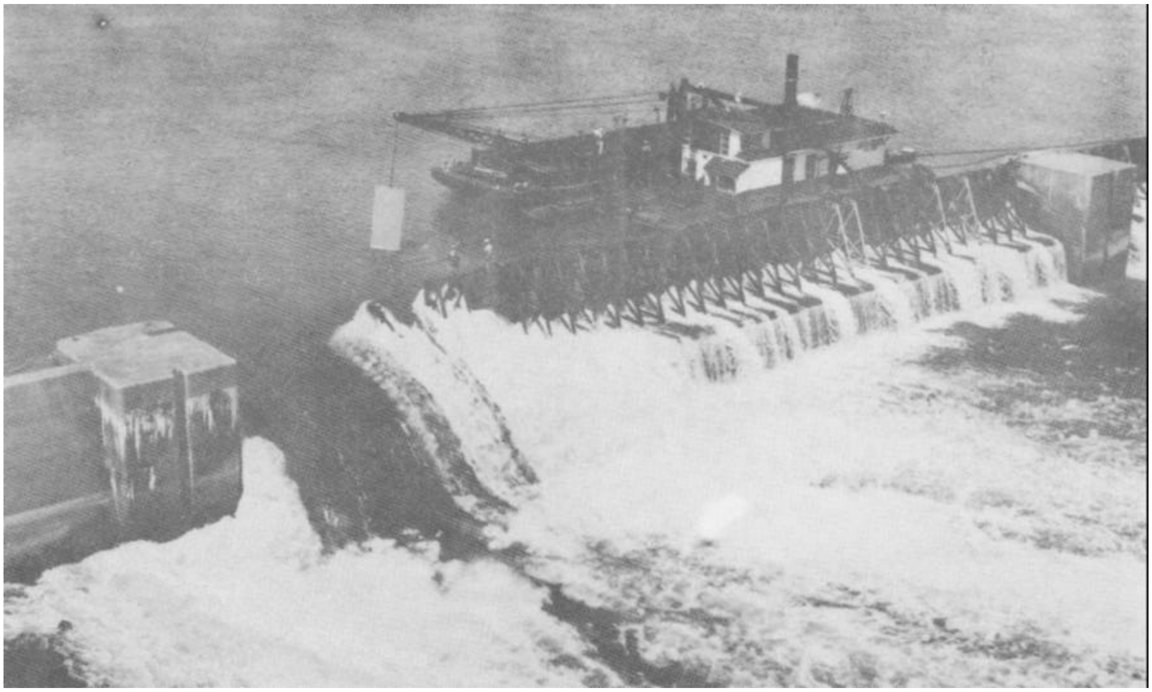


BOULE' DAM

Figure 5-18. Typical movable dams (Sheet 1 of 2)



Chanoine Wicket



Boule Dam
Figure 5-18. (Sheet 2 of 2)

Some of the disadvantages encountered in the use of vertical-lift gates are:

(1) Heavier lifting load which requires greater hoist capacity and often necessitates a “split-gate.” The split-gate increases operation difficulties.

(2) Not favorable for adaption to fixed individual hoist operation. The most common method of operation is by gantry crane which may have a greater first cost than do fixed hoists and also requires more operating personnel.

(3) Greater time required for gate operation because normally only one crane is provided. Time element may be especially significant at sites subject to flash floods.

(4) Gate slots lead to potential cavitation and debris collection.

5-14. Tainter Gate Design. Reference is made to EM 1110-2-2702 and EM 1110-2-1603 for design guidance for tainter gates. Additional design guidance is given in the following paragraphs.

a. Gate Seal Design and Vibration. Many laboratory and field studies have been concerned with instabilities (gate vibration and oscillation) at CE projects. Reports given in items 4, 7, 8, 17, 19-21, 23, and 24 of Appendix A are representative of problems encountered and their solution. The following guidance is recommended for gate seal design:

(1) The configuration of the tainter gate lip and bottom seal is a major factor in setting up flow conditions that cause gate vibrations. Ideally, tainter gate lips should provide as sharp and clean a flow breakoff point as possible. Supporting structural members downstream from the lip should be kept as high and narrow as possible. The Type C gate lip design (Figure 5-19), as used on Arkansas River Locks and Dams 8, 9, 13, and 14 gates, adequately meets these criteria. Severe vibrations adequate to eventually destroy the gates were experienced with Types A and B (see item 21, Appendix A).

(2) Rubber seals should not be used on the gate bottom unless water conservation requirements cannot tolerate the normal leakage. If required, a narrow rubber bar seal attached rigidly to the back side of the gate lip, as in type D design (Figure 5-19), is recommended. However, even minor variations from this seal design can result in vibrations. Consideration should also be given to providing a rubber seal in the gate-sill bearing plate. However, such seals are normally more difficult to maintain than gate-mounted seals.

(3) In wider tainter gates with high trunnion anchorages, the hydrostatic force of the pool against the skin plate tends to bow up the lip at the center of the gate. The Type D seal designs are too inflexible to prevent leakage under these conditions. The Type A designs are very flexible but also vibration prone. Figure 5-20 shows an untested lip design developed to prevent this leakage problem. The notch in the gate sill may be subject to cavitation damage and should be tested under proposed operation conditions before being adopted.

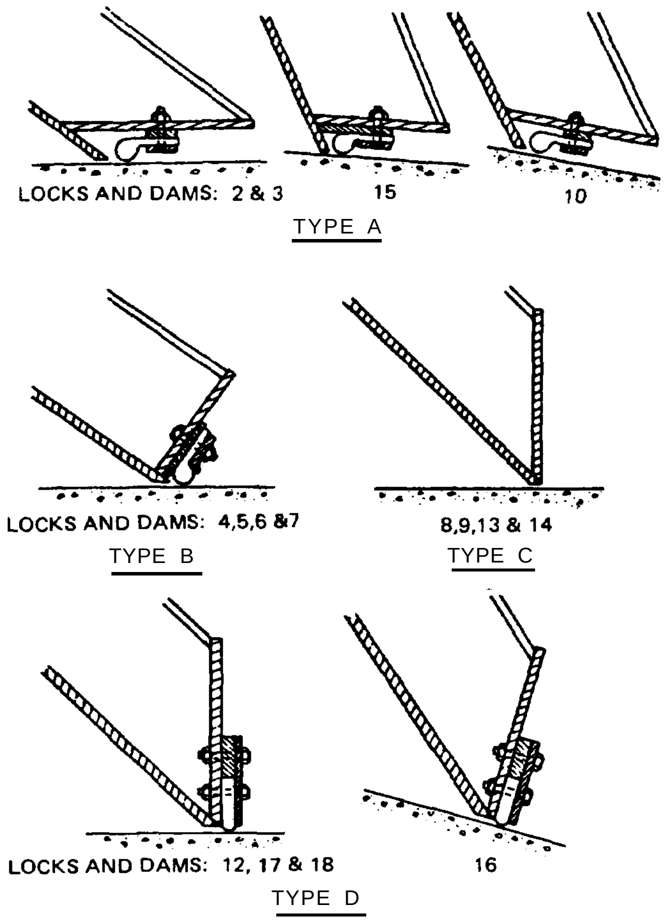


Figure 5-19. Gate lip design

(4) Structurally, the gate members should be rigidly designed to limit possible gate flexing under hydraulic loads. Rigid rib-to-girder welded connections and stiffener braces between the bottom girders and the cantilevered portion of the skin plate provided the necessary rigidity on the Arkansas gate designs.

(5) Gate side seals should be designed with sufficient flexibility to remain in contact with the side seal plates at all gate openings and for all probable gap openings as might be caused by construction inaccuracies, gate skews, gate temperature shrinkage and expansion, and normal structural settlements. The side seals should initially be set with a slight deflection forcing the seal against the seal plate. Debris that becomes wedged between the seal and seal plate should be cleaned out at regular intervals. The normal J-bulb gate side seal is shown in Figure 5-21. Also shown is a modified rubber seal shape that was designed to maintain a seal over wide gap variations between the gate and the pier. This design should be tested on a prototype gate before extensive use on proposed projects.

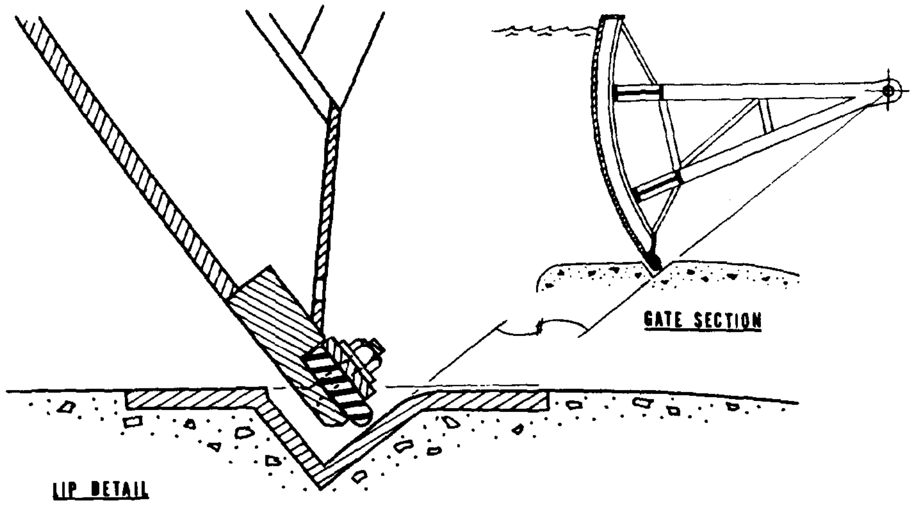


Figure 5-20. Bottom seal design for tainter gates, design proposed for vibration-free, leakage-free operation

(6) Unusual gate designs or features should be tested in model facilities or, if practical, on existing spillway gates that have similar geometric and hydraulic conditions to ensure against cavitation tendencies.

(7) No spillway tainter gate design or feature should be predicated, or made contingent, on the use of any specific gate operating scheme or plan.

b. Surging of Flow. Design criteria have been developed to prevent periodic surging of flow on spillway tainter gates. Model tests have indicated that the most effective means of eliminating the periodic surge on the tainter gates is to decrease the length of crest piers upstream from the gates or to increase the width of gate bays, or both. For low-overflow spillways, the gate-bay width should be equal to or greater than:

(1) 1.1 times the maximum head on the weir crest for which the gates control the discharge when the length of crest piers is less than 0.3 times the gate-bay width.

(2) 1.25 times the maximum head on the weir crest for which the gates control the discharge when the length of crest piers is between 0.3 and 0.4 times the gate-bay width. The maximum gate opening for which tainter gates will control the discharge should be taken as 0.625 times the head on the weir crest. By utilizing the spillway discharge curves for various gate openings, the maximum head on the weir crest for which the gates will control the discharge can be determined.

c. Gate Seat Location. The gate seat should be located at the beginning of the parabolic drop or within two feet upstream of that point for low-head navigation structures. This location will help the jet adhere to the downstream face of the crest.

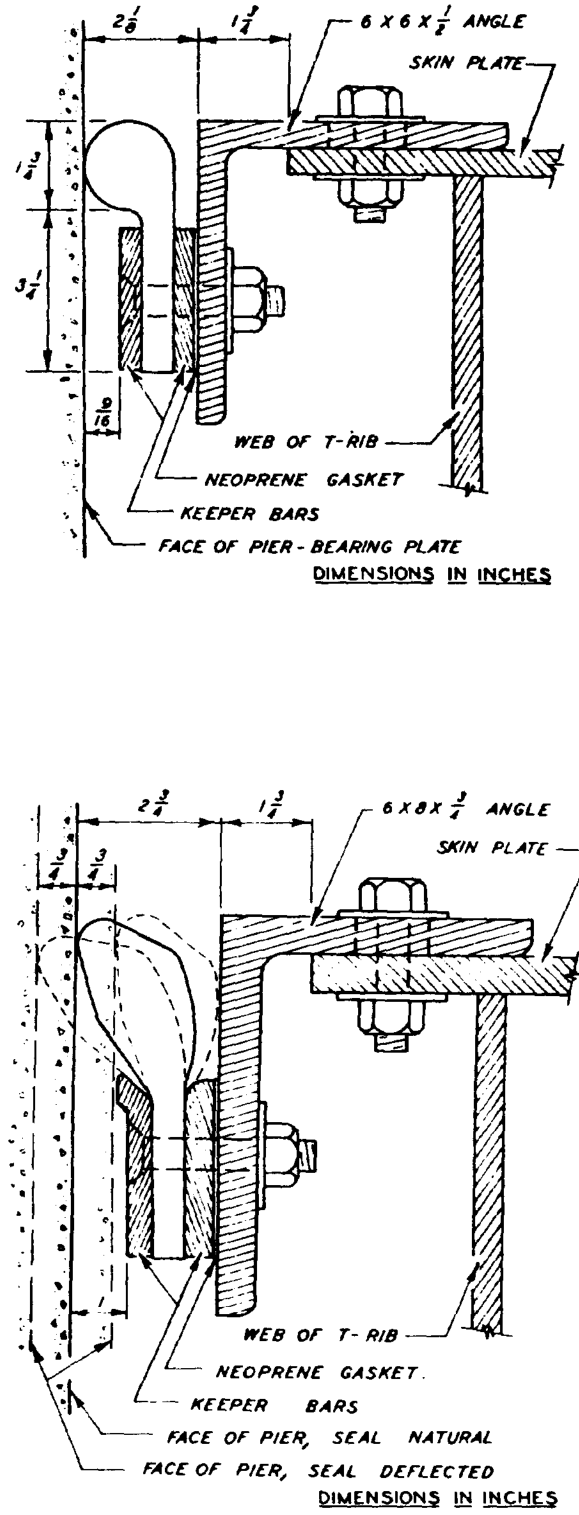


Figure 5-21. Gate side seals

d. Tainter Gate Trunnion Elevation. Trunnion elevation is set above most floods. Typical submergence allowed is a maximum of five to ten percent of the time.

e. Top of Gates, Closed Position. When in the closed position, the gates should have at least one foot of freeboard above the normal upstream pool. On large pools where fetch for wave setup is large and water conservation is important more than one foot may be required.

f. Bottom of Tainter Gates, Raised Position. Gates should be designed to clear the highest flood with allowance for floating debris. Typical clearance is one to five feet above the PMF. Special consideration may be appropriate for projects with major flood levees along the overbanks. Often the maximum stage will occur just before the levees are overtopped. Subsequent discharge increases would result in lowered stages because of levee failure and dispersion of flows through the protected areas. For spillways in such locations, the maximum gate-opening height would be set at one foot above the adjacent levee crown elevation. Another consideration is raising the bottom of the gates to allow accidental passage of barges through the gate bays without damage to the tainter gates.

g. Gate Radius. Skin plate radius ranges from 1.0 to 1.2 times the damming height of the gate. The radius of the gate is affected by the vertical distance between the bottom of the gate in the lowered position and low steel of the gate in the raised position. Spillway bridge clearance may also be a factor in determining the gate radius and the trunnion location.

h. Submergible Tainter Gates. Submergible tainter gates were developed to allow passage of ice without having to use large gate openings. Case histories of various types of submergible gates are presented in item 30 of Appendix A. Two types have evolved, the type in which the top of the gate can be lowered below the normal upper pool elevation and the piggyback gate. Both types are shown in Figure 5-3. A shaped lip on the top of the gate can be used to keep the flow off the back of the gate. A listing of projects having submergible tainter gates is given in Table 5-4 and a definition sketch is shown in Figure 5-22. Some of these projects have experienced scour and/or vibration problems. Lifting chain or cable loads are much greater in deep submerged positions and must be considered in machinery costs. At Lock 24, Upper Mississippi, submerged tainter gates have only been effective for passing light floating ice.

5-15. Vertical-Lift Gate Design. Reference is made to EM 1110-2-2701 and EM 1110-2-1603 for design of vertical-lift gates.

5-16. Spillway Piers. The hydraulic performance and discharge capacity of spillways are affected by the pier designs. The following factors need to be considered.

a. Thickness. Pier thickness is dependent upon structural requirements and is generally a function of the bay width and pier height. Pier widths for the spillways of item 10 and 13-16 projects, Appendix A, vary from 8 to 15 feet.

TABLE 5-4

Projects with Submergible Tainter Gates*

Lock and Dam (COE District)	River	H_D	Sub_G	C_G	D_G	Problem	Remarks
Greenup (Huntington)	Ohio	32.0	7.0	28.0	25.0	Yes	Problem: stilling basin and sill erosion and vibration Solution: submerged operation eliminated, plans and specs for modification as of Dec 1978
Meldahl (Huntington)	Ohio	30.0	7.0	28.0	23.0	Yes	Problem: stilling basin and sill erosion and vibration Solution: submerged operation eliminated
Markland (Louisville)	Ohio	34.0	7.0	33.0	27.0	Yes	Problem: vibration and jet through stilling basin and across end sill Solution: gate stops added and spillway curve modified. Submerged operation eliminated
McAlpine (Louisville)	Ohio	37.0	7.0	12.0	30.0	Yes	Problem: bed-material abrasion of sill Solution: submerged operation eliminated, spillway curve modified
Cheatham (Nashville)	Cumberland	26.0	7.0	19.0	19.0	Yes	Problem: vibration Solution: submerged operation eliminated, design modification being considered
New Cumberland (Pittsburgh)	Ohio	22.6	7.0	12.5	15.6	Yes	Problem: vibration, cavitation between gate and sill, and recreational craft hazard Solution: submerged operation eliminated
Pike Island (Pittsburgh)	Ohio	21.0	7.0	20.0	14.0	Yes	Problem: excessive leakage Solution: submerged operation eliminated
L&D No. 4 (Pittsburgh)	Monongahela	16.6	7.0	12.5	9.6	No	Movable crest or piggyback gate
Maxwell (Pittsburgh)	Monongahela	19.5	7.0	19.0	12.5	No	Movable crest or piggyback gate
L&D No. 11 (Rock Island)	Mississippi	11.0	8.0	12.0	3.0	No	13 gates
L&D No. 12 (Rock Island)	Mississippi	9.0	8.0	12.0	1.0	No	7 gates
L&D No. 13 (Rock Island)	Mississippi	11.0	8.0	12.0	3.0	No	10 gates
L&D No. 16 (Rock Island)	Mississippi	9.0	8.0	12.0	1.0	No	3 of 15 gates
L&D No. 17 (Rock Island)	Mississippi	8.0	8.0	8.0	0.0	No	8 gates
L&D No. 18 (Rock Island)	Mississippi	9.8	5.0	15.0	4.8	No	14 gates
L&D No. 20 (Rock Island)	Mississippi	10.0	3.0	17.0	7.0	No	6 of 40 gates
L&D No. 21 (Rock Island)	Mississippi	10.5	8.0	12.0	2.5	No	10 gates
L&D No. 22 (Rock Island)	Mississippi	10.5	8.0	17.0	2.5	No	1 of 10 gates

(Continued)

TABLE 5-4 (Concluded)

Lock and Dam (COE District)	River	H_D	Sub_G	C_G	D_G	Prob-lem	Remarks
St. Louis L&D No. 24	Mississippi	15.0	8.0	17.0	7.0	Yes	15-80' TG's, vibration, stress on trunion; submerged operation eliminated
St. Louis L&D No. 25	Mississippi	15.0	7.0	18.0	9.0	Yes	14-60' TG's, vibration stress on trunion; submerged operation eliminated
St. Louis L&D No. 26	Mississippi	24.0	3.0	27.0	21.0	No	30-40 TG's

* See Figure 5-22 for definition sketch.

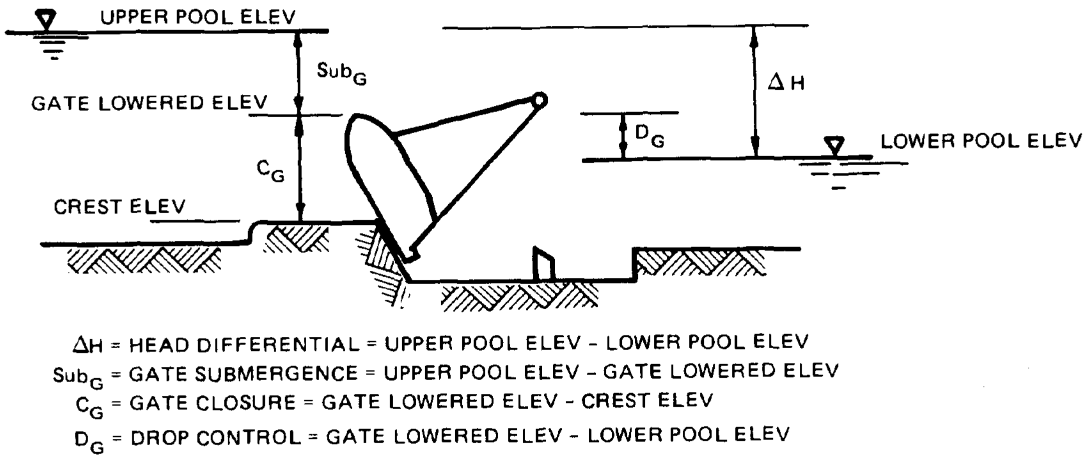


Figure 5-22. Definition sketch for variables used in Table 5-4

b. Supplemental Closure Facilities. Bulkheads are provided on all gated navigation spillways to permit gate maintenance without draining the pool. Bulkhead slots are located in the piers and have their upstream side about one pier thickness downstream from the pier nose. The slots must be upstream far enough to ensure that the bulkheads will clear the gate raising mechanisms while being placed. Occasionally, bulkhead slots are provided on the downstream ends of piers also. These bulkheads would permit dewatering and inspection of the spillway gate sill. When lower pool levels are higher than the gate sill, inspections must be made by divers if these bulkheads are not provided.

c. Pier Nose Shape. A semicircular pier nose shape is the most common and generally satisfactory design. An ogival shape (Type 3, HDC 111-5) was found to be only slightly more efficient than the semicircular shape (see item 6, Appendix A). All the Arkansas River navigation spillways have a curved nose leading to a 90-degree point (similar to ogival). A structural angle is embedded in the point. The angle has helped to protect the piers from being damaged by colliding barges and other objects. This shape is very efficient when the gates on both sides of the pier are set at equal openings. However, when gate settings are very different, the sharp pier nose causes a flow separation from the pier on the larger gate opening side causing a reduction in efficiency.

d. Barge Hitches. If floating plant is used for spillway or spillway gate maintenance, tie-up posts should be added to both the upstream and downstream end of the piers. By recessing the posts back from the pier face, they will cause minimal flow disturbances.

5-17. Abutments. Long-radius abutments are used infrequently at low-head navigation dams because the spillway is normally located for straight approach flow which minimizes need for large abutments, and operation of adjacent locks, overflow sections, powerhouses, etc., would be hindered by large abutments. Abutment radius used on projects in items 10 and 13-16 of Appendix A were the same as the interior piers that equaled one-half of the pier width.

Section II. Design of Other Appurtenances

5-18. Navigable Passes. Navigable passes permit the passage of tows over low head dams without the requirement for locking. These may be appropriate at some dams if certain conditions obtain. These include stages high enough to permit open-river navigation for a significant portion of the year, individual high-water periods usually of considerable duration, and a gate regulating system commensurate with the rate of river rise and fall. The benefits of a navigable pass may include lower lock wall heights and lower tow operating costs when lockage is unnecessary. This may be offset by higher maintenance costs for locks that sustain relatively frequent overtopping. In addition to dams for which a navigable pass is included as an element in their configuration, many other dams have high-water navigation over a weir section. This includes both dams with gated and weir sections as well as dams entirely constructed as fixed-crest structures. These dams also may require less lock-wall height. The design of a navigable pass must provide for sufficient clear width for safe passage of tow traffic, including poorly aligned tows. At some locations this may include two-way traffic. In addition, the pass must have sufficient depth for tows of the authorized draft, including a buffer to account for overdraft, tow squat, etc. Model studies have shown that a navigable pass should have a minimum cross-sectional area 2-l/2 times the area blocked by a loaded tow. Current direction should be aligned normal to the axis of the navigable pass and velocity through the pass must be low enough for upbound loaded tows of the horsepower range that operates on the waterway. A model study should be considered in the design of a navigable pass. At the present time, the Corps is operating dams with navigable passes on the Ohio and Ouachita Rivers. Pass widths vary from 200 feet on the Ouachita to 932 and 1,248 feet on the Ohio River. In addition, the Corps operates dams on the Illinois Waterway at which tows transit the regulating wicket section during higher stages. Gate types for navigable passes include Chanoine wickets (Figure 5-18) and hydraulically operated bottom hinged gates. Fabridam has also been used but has experienced considerable problems with vandals and debris punctures. Drum gates are under consideration for a replacement structure on the Ohio River (Figure 7-3).

5-19. Low-Flow and Water Quality Releases. Provision for sluices as part of the main spillway or a separate outlet works to accomplish low-flow or multilevel releases should be designed according to EM 1110-2-1602.

5-20. Fish Passage Facilities. Most fish passage facilities are located on rivers in the North Pacific Division (NPD). Engineers in NPD should be contacted for design information.

5-21. Ice Control Methods. It is desirable and often essential to continue operation of navigation dams and spillways during winter. Traffic may be curtailed or even stopped on the waterway but provision must be made to pass winter flows and to handle ice during winter and at breakup. Designers must consider ice passage procedures, possible ice retention, ice forces on the structures, and icing problems leading to blocking of moving parts or simply excess weight (Figure 5-23). Provisions to move ice past or through dams have

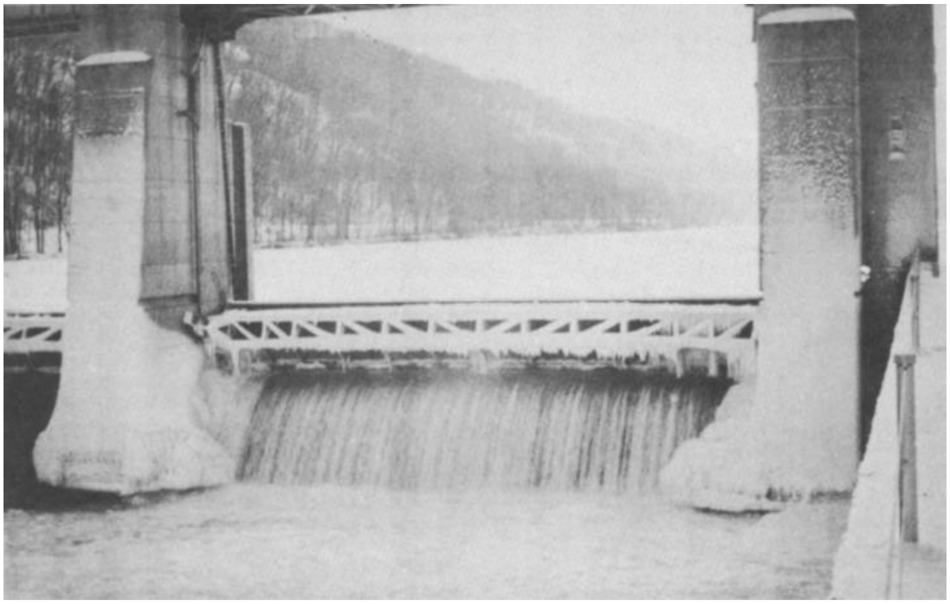


Figure 5-23. Ice on control gate

been many and varied and none have met with perfect success. At some locations, it is preferable to retain the ice in the upstream pool, while at others an ice-passing capability is necessary. Spillway gates should be as wide as practicable to minimize arching across the openings. The primary factor controlling ice passage appears to be the velocity of the approaching ice. When the velocity is great enough, the flows are broken and pass through spillway bays. Passage of ice through a submerged outlet requires sufficient velocity to entrain the ice into the flow. Therefore, to maintain pool during periods of low flow, it is preferable to pass ice over the top of gates in a skimming type mode. At low flows ice can be passed with one or more gates open at a time and arching broken by alternating gate openings. Physical models of ice control methods for specific projects can be made in the Ice Engineering Laboratory at the Corps of Engineers Cold Region Research and Engineering Laboratory in Hanover, N. H. EM 1110-2-1612 provides additional information on ice control methods.

Section III. Model Studies

5-22. General.

a. In the design of navigation dam spillways for major structures, a combination of analytical, laboratory, and field studies is usually needed. The laboratory studies can be physical or numerical models of flow conditions which are usually conducted at WES or ice studies for dams in cold regions which can be modeled at the Ice Engineering Laboratory at CRREL. Numerous problems in the design of spillways cannot always be solved satisfactorily without the use of model studies. Experience has shown that a model often can indicate more economical treatment of certain features which may reduce construction costs by many times the cost of the model. A model may reveal inadequacies in the basic design that would limit discharge capacity, result in costly maintenance, or even cause hazardous operation. It may be desirable to use hydraulic models for a specific project or for a typical case of a number of small structures. By using model studies, alternate plans and modifications can be tested within a relatively short time with all flow conditions that can be expected. Also, the design and operating engineers can observe conditions resulting with a particular arrangement and satisfy themselves as to the adequacy of the plan in addition to the advantages given above.

b. Examples of previous hydraulic models at WES used to solve spillway design problems are numerous. Among the most common usages is the verification of general spillway adequacy and performance. Generally, undistorted models of various linear scale ratios are used (commonly 1:12 to 1:60) depending upon the problems involved, and practical space and discharge limitations. A general model is normally used when approach conditions, flow over the spillway, and exit channel hydraulics are to be studied. A section model simulating one or more spillway gate bays is extremely effective for improving various details of spillway design at larger scales than the general model. If only a section model is to be used to simulate a structure, careful consideration should be given to the model limits since a two-dimensional model may not introduce flow patterns that can be addressed in a three-dimensional model.

c. The effect of approach conditions on discharge of a navigation dam spillway and required excavation can be studied to advantage in a model. Abutment configuration may seriously affect the discharge of a spillway, and the model can indicate the most cost-effective design. The effect of waves from the ends of piers upon the height of sidewalls can best be studied in a model.

d. Determination of the performance of stilling basins is an important objective in hydraulic model studies. The length and width of stilling basins and the arrangement of baffles and end sills can be tested. The scour tendency and protective measures downstream from stilling basins can also be studied in a model.

e. A typical example of model study benefits is found in item 13 of Appendix A, where tests of a spillway as originally designed indicated that several modifications could improve performance and reduce project cost. Stilling basin tests demonstrated that the apron could be raised two feet to el 87.0 and still maintain an adequate jump under the most critical operating condition of one gate fully opened with the normal pool and minimum tailwater elevation expected. Two rows of baffles, eight feet high, seven feet wide, and eight feet apart, were found to be more beneficial than the original single row in dissipating energy and maintaining the hydraulic jump. Pier extensions 37 feet long and 23 feet high were essential for the elimination of return flows and eddies experienced during single-gate operations. A lower terminal apron elevation and riprap on a 1V-on-20H upslope were required downstream of the stilling basin to prevent the formation of a secondary jump over the horizontal downstream riprap protection. Multiple- or single-gate openings greater than six feet created a secondary jump with the original design basin and low tailwaters. The recommended design stilling basin eliminated the secondary jump and provided satisfactory energy dissipation for both normal and emergency operating conditions. Other changes from the original design included eliminating the approach trench upstream of the spillway, eliminating the go-degree curved endwall downstream of the left stilling basin training wall, and shortening the right training wall between the gated and ungated spillways from 115 to 40 feet. The approach trench was removed to prevent irregular flow conditions. The go-degree curved endwall tended to magnify wave action on the left bank. Reducing the length of the right training wall was economically beneficial since any length beyond 40 feet did not improve hydraulic performance. A considerable reduction in the excavation requirements along the right downstream bank was recommended to improve flow patterns and decrease construction costs. This recommended reduction in width decreased eddy action, eliminated much of the return flow along the right bank, and produced better flow patterns for both single- and multiple-gate operations.

Section IV. Example Design

5-23. <u>Known Information.</u> From optimization study (see Appendix D for example), a six-gated structure is required having the following dimensions:

Normal Upper Pool Elevation = 140

Normal Lower Pool Elevation = 110

Crest Elevation = 100

Maximum High Water Elevation = 165

Tailwater Stage Exceeded 10 Percent of the Time = 139

Tailwater Buildup Is Slow

Channel Invert Elevation = 100

Left Side of Spillway Adjacent to Lock Wall

Right Side of Spillway Has 1V-on-3H Side Slope

Use Standard, Nonsubmergible Tainter Gate

Gate Width = 60 feet = (Width of Monolith - Pier Width)

Pier Width = 10 feet

Unit Weight of Available Stone = 165 lb/ft^3

Riprap to be Placed in the Dry

5-24. Development of Design.

a. Upstream Face and Radius - Use vertical upstream face with a five-foot radius (due to 40-foot head) connecting the upstream face and horizontal crest.

b. Structural requirements usually dictate length of horizontal crest from upstream face to beginning of downstream face. Past projects have used approximately 110 percent of the head on the crest. Distance = 1.10(40) = 44 feet.

c. Downstream Face:

H = Normal Pool - Crest Elevation = 40 feet

V_o (for parabolic drop) = $\sqrt{2g(1/3)H}$ = 29.3 ft/sec

$$X^2 = \frac{2V_o^2Y}{g} = \frac{2(29.3)^2Y}{32.2} = 53.3Y \qquad \text{(5-l bis)}$$

This is the steepest slope recommended for a head of 40 feet; use X^2 q 55Y. The downstream face shaped according to this equation will not experience severely negative pressures and the jet will adhere to the downstream face of the crest. Point at which slope equals 1V on 1H:

$$Y = \frac{X^2}{55}$$

$$\text{Slope} = \frac{dY}{dX} = \frac{2x}{55}$$

$$\text{For slope} = 1 = \frac{2x}{55}\ ,\ X = 27.5\ ,\ Y = 13.75$$

d. Discharge Rating - Free uncontrolled flow is needed for input into stilling basin design. Some of the other three flow regimes require the stilling basin apron elevation and will not be computed in this step.

$$Q = C_F L H^{3/2} \qquad (5\text{-}2\ \text{bis})$$

Using Figure 5-9, and using an abutment contraction coefficient since the adjacent bays are not operating, the following table results for discharge through a single bay.

Upper Pool Elevation	H_e/R*	$K_a/2$**	$L_{effective}$, feet	H/B_c	C	Q, cfs/bay
100	0	--	60.0	0	--	0
105	1	0.015	59.85	0.11	3.00	2,007
110	2	0.021	59.6	0.23	3.04	5,730
115	3	0.027	59.2	0.34	3.07	10,557
120	4	0.036	58.6	0.45	3.09	16,196
125	5	0.04	58.0	0.57	3.11	22,548
130	6	0.042	57.5	0.68	3.15	29,762
135	7	0.044	56.9	0.80	3.19	37,584
140	8	0.046	56.3	0.91	3.24	46,163

* R = l/2 pier width for use in HDC 111-3/1
** See paragraph 5-7c

e. Stilling Basin Apron Elevation - Use a single gate fully opened, normal upper pool, and minimum tailwater (which equals the normal lower pool since there is a slow tailwater buildup) to determine the apron elevation. The unit discharge into the basin is

$$q = \frac{Q}{W} = \frac{46163}{60} = 769.4\ \text{cfs/ft}$$

<u>Assume</u> Stilling basin apron elevation = 75

Solve Equation 5-8 by trial and error for V_1 and d_1 using no energy loss between upper pool and stilling basin apron

$$140 = 75 + \frac{V_1^2}{2g} + d_1$$

$$V_1 = \frac{q}{d_1} = \frac{769.4}{d_1}$$

we are actually solving

$$140 = 75 + \frac{\left(\frac{769.4}{d_1}\right)^2}{2g} + d_1$$

The solution is $d_1 = 13.35$ feet

and

$$V_1 = \frac{769.4}{13.35} = 57.6 \text{ ft/sec}$$

$$F_1 = \frac{V_1}{\sqrt{gd_1}} = 2.78$$

$$\frac{d_2}{d_1} = 0.5\left(\sqrt{1 + 8F_1^2} - 1\right) = 3.46$$

$$d_2 = 3.46(13.35) = 46.2 \text{ feet}$$

Check assumed stilling basin elevation using tailwater equal to 85% d_2

(Factor = 0.85 in Equation 5-11)

$$110 - 75 \neq 0.85(46.2)$$

$$35 \neq 39.3$$

A new stilling basin apron elevation must be assumed until the above equation is satisfied. The correct solution is an apron elevation = 69.0.

$$d_1 = 12.55 \text{ feet}$$

$$V_1 = \frac{769.4}{12.55} = 61.31 \text{ ft/sec}$$

$$F_1 = \frac{61.31}{[(32.2)(12.55)]^{1/2}} = 3.05$$

$$d_2 = 48.25 \text{ feet}$$

f. Basin Length - Distance from beginning of basin to 1V-on-5H upslope $L_2 = 2d_1F_1^{1.5} = 133.7$ ft.

g. Baffles - Height = $0.25d_2$ = 12.06, use 12 feet. Distance to first row = $1.3d_2$ = 62.7 feet. Distance between upstream faces of baffle = 2(12) = 24 feet.

h. Pier Extensions - Extend 57.7 feet into basin. Use five feet wide beyond main piers and use top elevation of 112 (two feet above lower normal pool).

i. End Sill - Use end-sill height = $0.15d_2$ = 7.2 feet, use 7.0.

j. Training Wall - Extend right training wall to end of basin at a top elevation of 112.

k. Approach Area Configuration - Use approach five feet below crest, horizontal for 50 feet, and slope up to streambed for 100 feet at 1V on 20H.

l. Approach Area Riprap - Average velocity = 769.4/(140 - 95) = 17.1 ft/sec. Using Equation 5-13, we have the following choices:

C	Thickness in D_{100}	Gradation Table	D_{50}(MIN), feet	W_{50}(MIN), lbs	Thickness inches
0.44	1.0 D_{100}(max)	5-2	1.4	258	30
0.30	1.5 D_{100}(max)	5-3	1.0	86	33

Gradations other than those given in Tables 5-2 and 5-3 could be used by determining D_{30} in Equation 5-13 with a blanket thickness of 1.0 D_{100}MAX).

m. Exit Channel Configuration - The top of the end sill will be at 69 + 7 = 76.0. Place top of riprap 1.0 foot below top of end sill. Slope exit channel up to streambed for 500 feet at 1V on 20H.

Exit Channel Riprap - Velocity over end sill w/o spreading = (769.G/llO - 76) = 22.6 ft/sec, use 0.80(22.6) = 18.1 ft/sec in Equation 5-14.

$$D_{50}(MIN) = 2.5 \text{ feet}$$

$$W_{50}(MIN) = 1,302 \text{ pounds}$$

Using gradation Table 5-3 for high turbulence, use thickness = 78 inches immediately below end sill.

Distance, feet	Thickness, inches
$3d_2$ = 150	78
$3d_2$ = 150	66
$2d_2$ = 100	48
$2d_2$ = 100	33

Adjacent to the lock wall, spreading of the single gate fully opened will be inhibited and rock size cannot be decreased as rapidly as given in the above table. Use 78-inch thickness for the first 300 feet then 66-inch thickness for the remaining 200 feet. Provide trench of riprap at downstream end to protect toe.

o. Tainter Gate Design - For this example design, a gate radius of 1.25 times the damming height of the gate will be used. In reality, this radius can depend on other factors not considered in this example. The trunnion elevation will be placed one foot above the stage that is exceeded 10 percent of the time.

$$R = 1.25(40) = 50 \text{ feet}$$

$$\text{Trunnion elevation} = 139 + 1 = 140 \text{ feet}$$

The gate seat location will be at the beginning of the parabolic drop.

p. Pier Design - Use semicircular pier noses located in the same plane as the upstream face of the structure.

q. Abutments - Abutment radius should be one-half the pier width or five feet.

r. Discharge Rating -

(1) Submerged Uncontrolled - Use the d'Aubuisson equation (5-5) with K = 0.90 since bay width = 60 ft. An iterative solution is required.

H, feet	h, feet	Approach Area, ft^2	K	AH, feet	Q, cfs All Gates
12.5	10	6,409	0.9	2.5	37,750
11.43	10	5,991	0.9	1.43	29,007
10.53	10	5,642	0.9	0.53	17,954
10.15	10	5,495	0.9	0.15	9,629
25.0	20	11,550	0.9	5.0	109,550
22.86	20	10,637	0.9	2.86	85,009
21.05	20	9,875	0.9	1.05	53,004
20.30	20	9,562	0.9	0.30	28,743
37.50	30	17,159	0.9	7.50	201,805
34.29	30	15,674	0.9	4.29	157,164
31.58	30	14,444	0.9	1.58	98,623
30.46	30	13,942	0.9	0.46	54,132

Results are plotted in Plate 5-1 along with the values for free uncontrolled flow.

(2) Free Controlled Flow - Using the coefficients presented in Figure 5-11:

H, feet	G_0, feet	C_g	Q, cfs/bay
30	1	1.0	2,636
30	6	0.69	10,912
30	14	0.58	21,401
20	1	0.90	1,937
20	6	0.65	8,393
10	1	0.82	1,248
10	6	0.54	4,930

Results are plotted in Plate 5-2 along with the curve for free uncontrolled flow. For heads greater than 30 feet or gate openings greater than 14 feet, HDC 320-4 to 320-7 must be used. The trunnion height above crest "a" equals 40 feet. This results in the ratio a/R = 40/50 = 0.8 which requires interpolation between HDC 320-5 and HDC 320-6. Determine L/P = 44/5 = 8.8 and find adjustment factor C2 = 1.03 .

H	G_o	G_o/R	H/R	C_1(a/R = 0.5)	C_1(a/R = 0.9)	C_1(a/R = 0.8)	Q, cfs/bay
30	20	0.40	0.60	0.495	0.528	0.520	28,230
40	20	0.40	0.80	0.517	0.555	0.546	34,230
40	14	0.28	0.80	0.545	0.605	0.590	25,138
40	6	0.12	0.80	0.622	0.723	0.698	13,130

(3) Submerged Controlled Flow - This type of flow requires a different rating curve for each gate opening. Using Figure 5-12 for c_{gs} , B = 31 feet:

G_o, ft	H, ft	h, ft	h/G_o	C_{gs}	ΔH, ft	Q, cfs/bay
1	30	10	10	0.076	20	1,636
1	25	10	10	0.076	15	1,416
1	20	10	10	0.076	10	1,156
1	15	10	10	0.076	5	818
1	30	20	20	0.037	10	1,126
1	25	20	20	0.037	5	796
6	30	10	1.67	0.47	20	10,114
6	25	10	1.67	0.47	15	8,759
6	20	10	1.67	0.47	10	7,152
6	15	10	1.67	0.47	5	5,057
6	30	20	3.33	0.23	10	7,000
6	25	20	3.33	0.23	5	4,950
14	30	20	1.43	0.58	10	17,652
14	25	20	1.43	0.58	5	12,482

Results are presented in Plate 5-3 along with the curves for free controlled flow.

s. Plan and profiles of the completed structure are given in Plates 5-4 to 5-6.

EXAMPLE DESIGN
FREE AND SUBMERGED
UNCONTROLLED FLOW

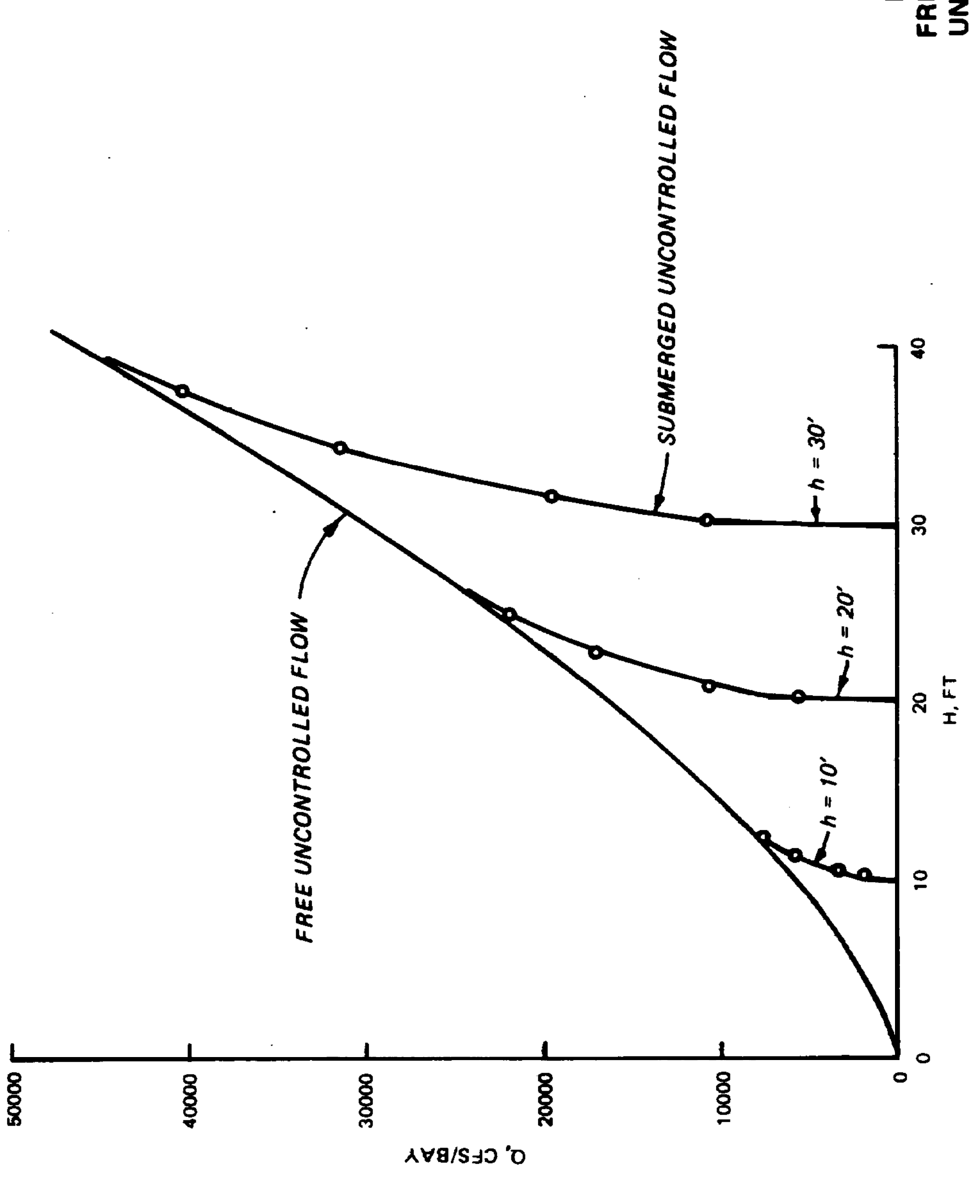
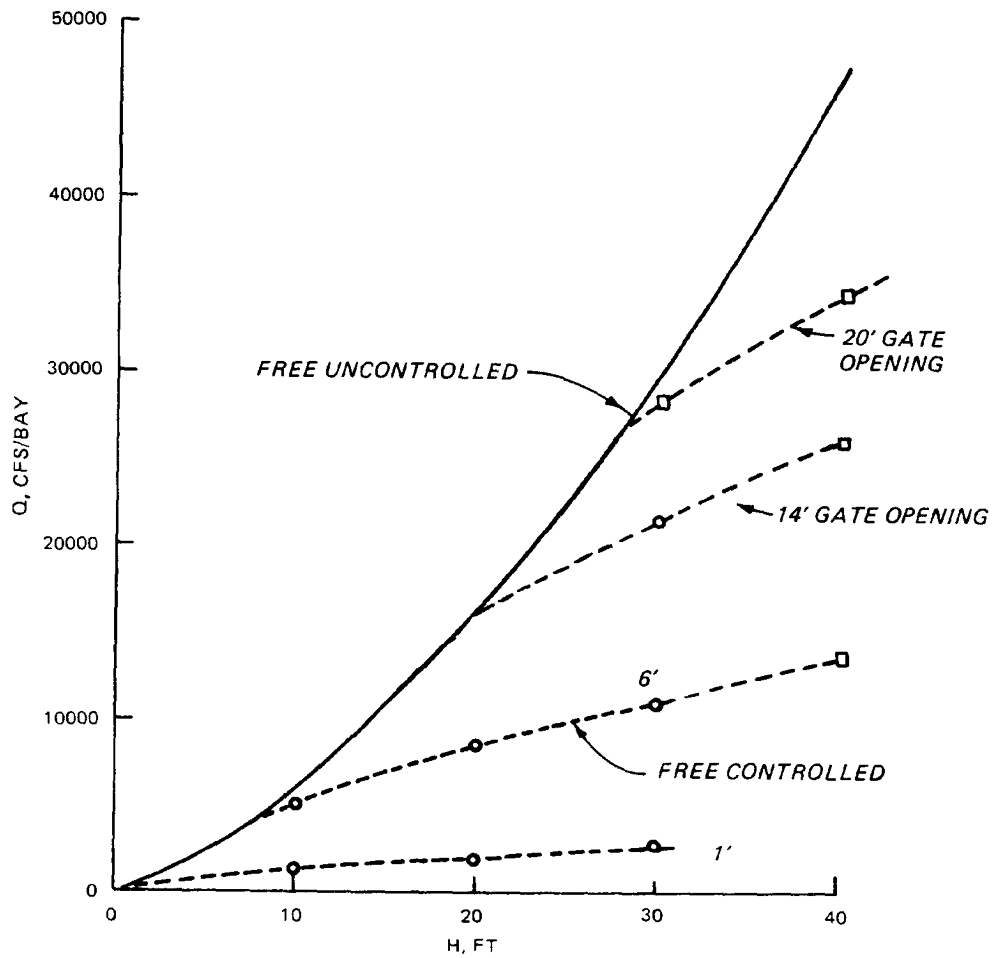


PLATE 5-1

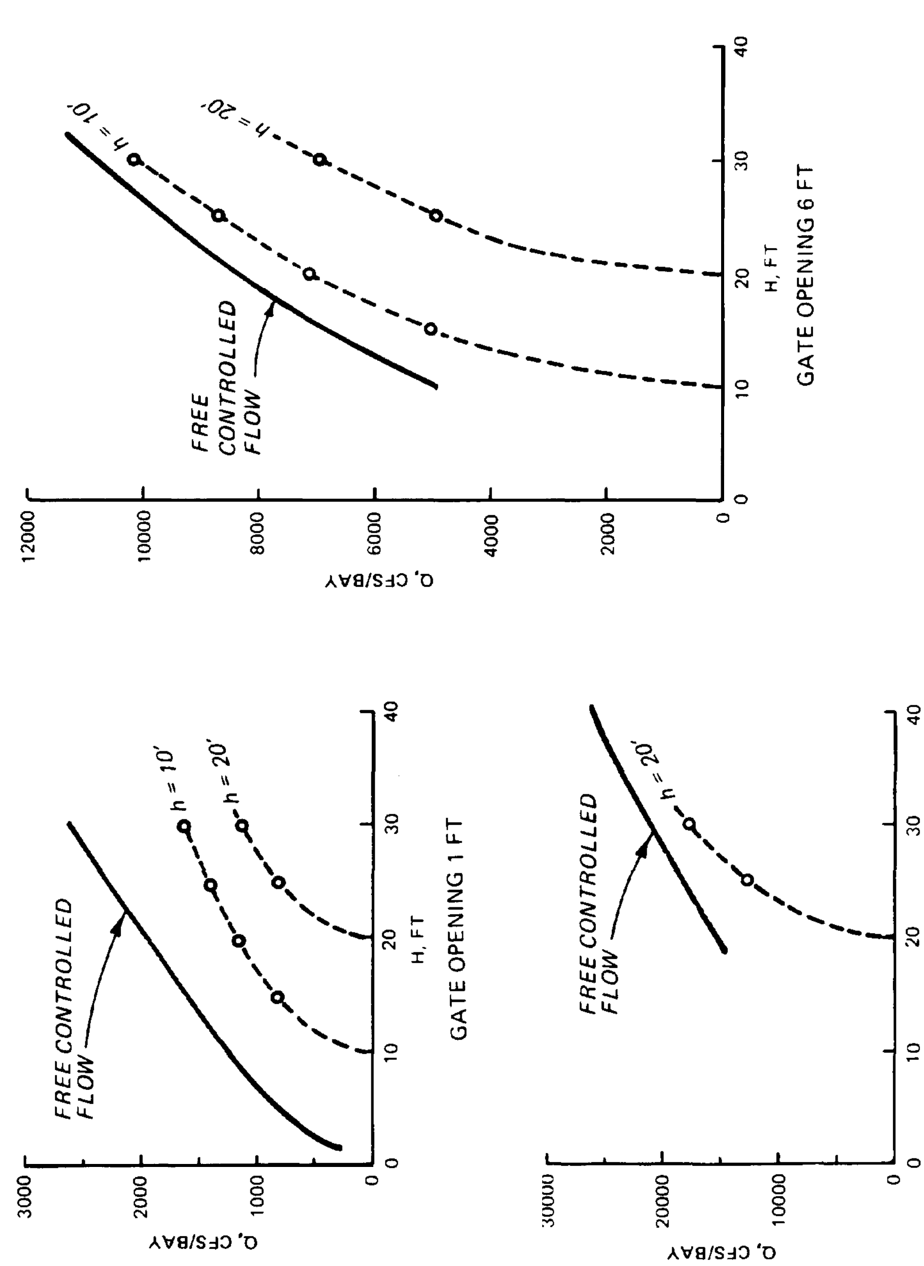


LEGEND

o FIGURE 5-11
◘ HDC 320-4 THRU 320-7

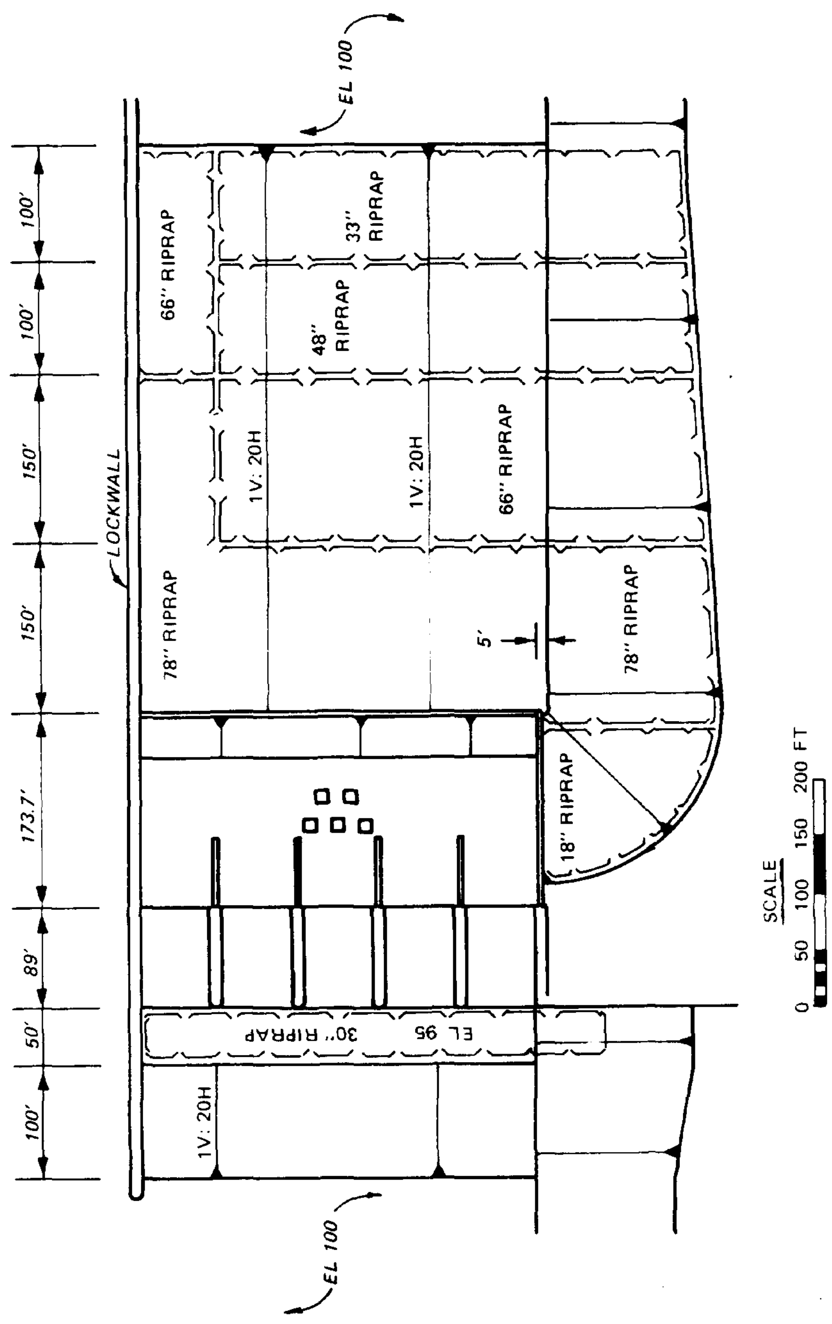
EXAMPLE DESIGN
FREE CONTROLLED AND
FREE UNCONTROLLED FLOW

PLATE 5-2



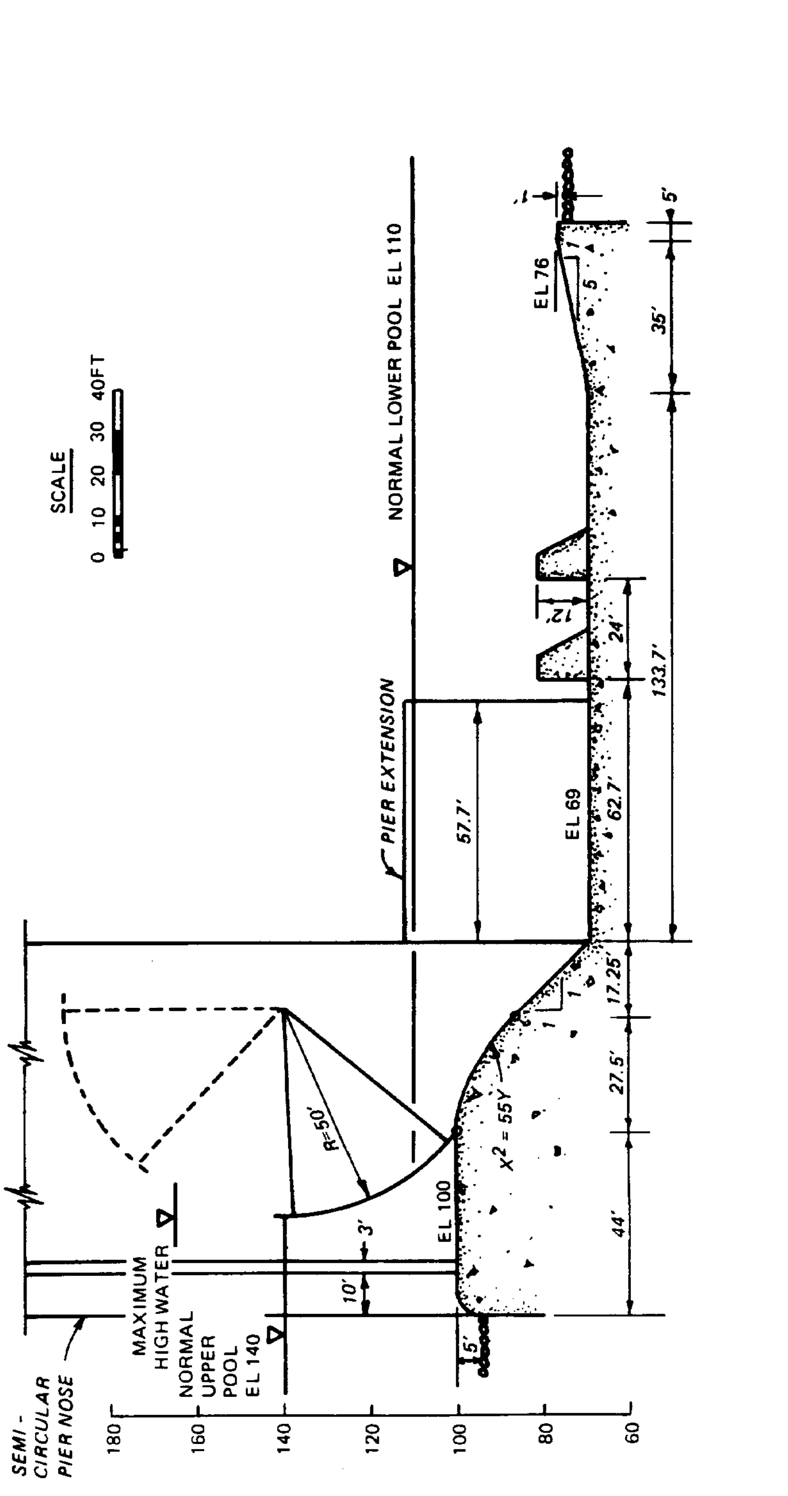
EXAMPLE DESIGN
FREE CONTROLLED AND SUBMERGED
CONTROLLED FLOW

PLATE 5-3



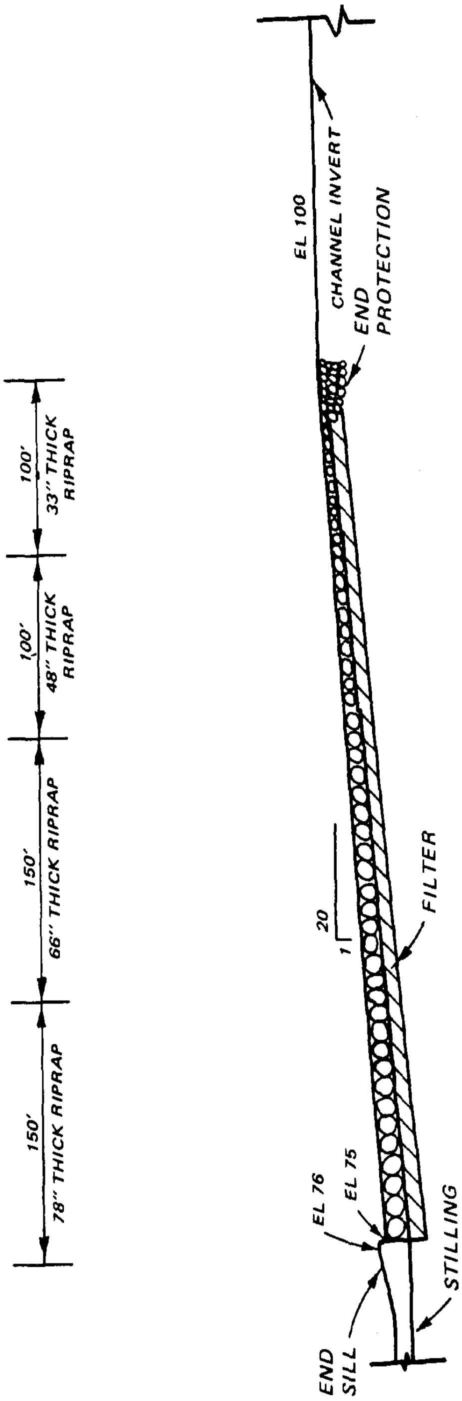
EXAMPLE DESIGN FOR FULL GATE OPENING PLAN VIEW

PLATE 5-4



EXAMPLE DESIGN FOR FULL GATE OPENING PROFILE 1

PLATE 5-5



**EXAMPLE DESIGN FOR
FULL GATE OPENING
PROFILE 2**

PLATE 5-6